Mattamy (Milton West) Ltd.

Framgard North and South Blocks

Stormwater Management Report

January 19, 2024

Framgard North and South Blocks

Stormwater Management Report

Mattamy (Milton West) Ltd.

Project No.: 231-00962
Date: January 19, 2024

WSP
100 Commerce Valley Drive West
Thornhill, ON
Canada L3T 0A1

T: +1 905 882-1100
F: +1 905 882-0055
wsp.com

Revision History

FIRST ISSUE

2023-02-17	Issued for Client Review	
Prepared by	Reviewed by	Approved by
Sukhjot Hans, Co-op Student	Gordon Wong, P.Eng., Project Engineer	Gordon Wong, P.Eng., Project Engineer

REVISION 1

2023-07-28	Rezoning Application (ZBA) – First Submission	
Prepared by	Reviewed by	Approved by
Arshia Mazaheri, Co-op Student	Gordon Wong, P.Eng., Project Engineer	Gordon Wong, P.Eng., Project Engineer

REVISION 2

2024-01-19	Rezoning Application (ZBA) – Second Submission	
Prepared by	Reviewed by	Approved by
Arshia Mazaheri, Co-op Student	Gordon Wong, P.Eng., Project Engineer	Gordon Wong, P.Eng., Project Engineer

FINAL

Prepared by	Reviewed by	Approved by

Signatures

Prepared by

01-19-2024

Arshia Mazaheri
Co-op Student, Water Resources

Date

Approved[1] by (must be reviewed for technical accuracy prior to approval)

Gordon Wong, P.Eng.
Project Engineer, Water Resources and Land Development

Date

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Contributors

Client

Mattamy (Milton West) Ltd.

WSP

Co-op Student, Water Resources	Arshia Mazaheri
Project Engineer, Water Resources	Gordon Wong, P.Eng.
Senior Project Engineer, Land Development	Wendy Cheung, P.Eng., PMP

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1 INTRODUCTION

1.1 Scope

WSP Canada Inc. (WSP) has been retained by Mattamy (Milton West) Ltd. to prepare a Stormwater Management (SWM) report to support the Rezoning Application (ZBA) for the proposed development, known as Framgard North and South Blocks, located in the Town of Milton in Halton Region. The address will be confirmed in a future submission.

This SWM report examines the potential erosion control, water quality, water quantity, and water balance impacts of the proposed development and summarizes how each will be addressed in accordance with the Town of Milton's stormwater management guidelines.

1.2 Site Location

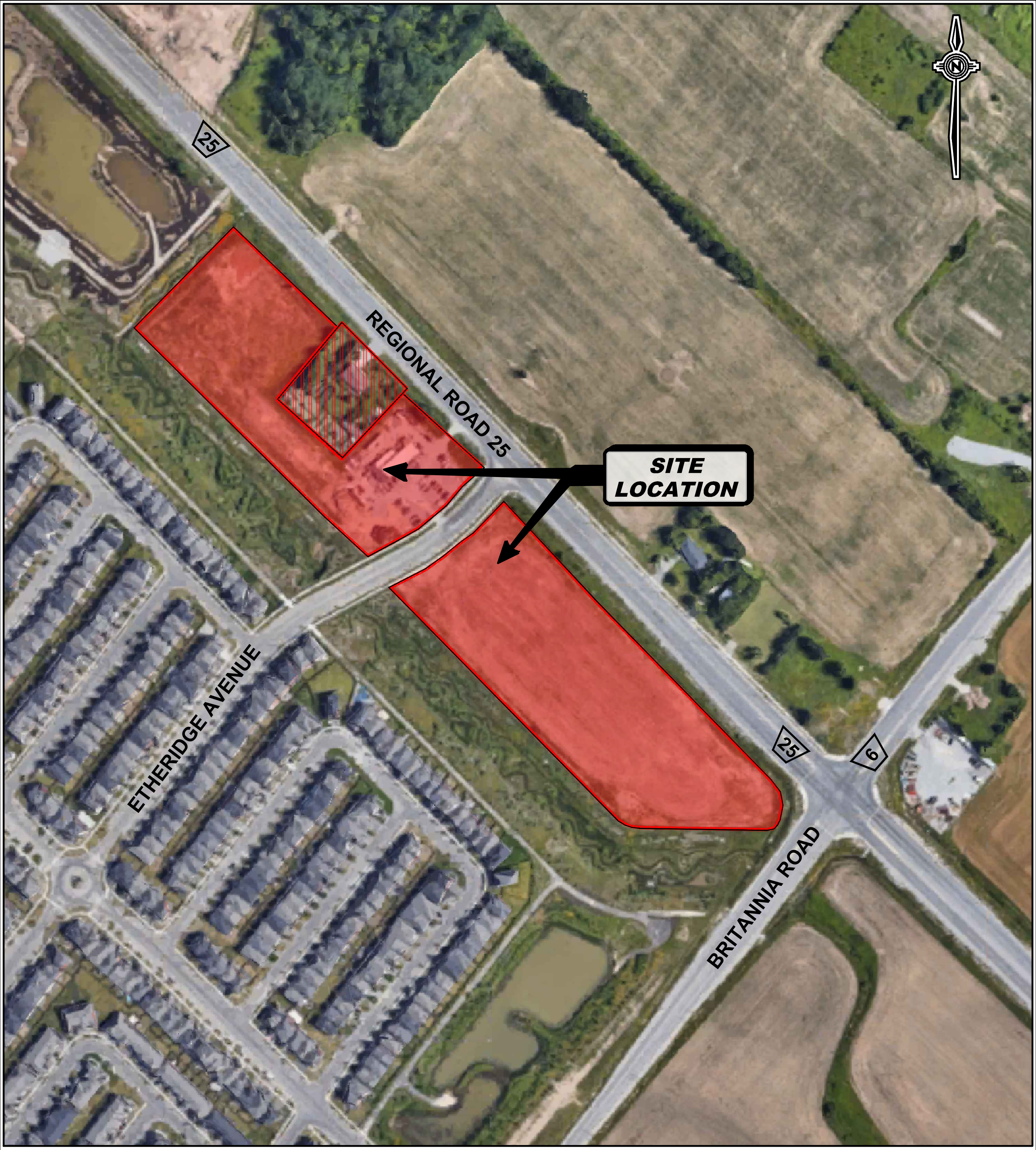
The site is located within the Town of Milton, in the Boyne Survey Block 2 Area 4. The site is bounded by an existing SWM pond, named SWM Pond "I", to the north, Regional Road 25 to the east, Britannia Road to the south, and an existing engineered channel, named SWS-2-A, to the west. Etheridge Avenue bisects the site and runs west to east separating the development into a South Block and a North Block. The site is an irregular shape due to a holdout property located in the North Block. The site is located within the Sixteen Mile Creek watershed which is part of the Town of Milton and Conservation Halton jurisdictions as the site is within the Conservation Authority regulated areas. The site is not within a highly vulnerable aquifer, or an area considered as a significant groundwater recharge area.

The location of the proposed redevelopment is illustrated in **Figure 1**.

1.3 Stormwater Management Plan Objectives

The objective of the Stormwater Management Plan are as follows:

- Determine site specific stormwater management requirements to ensure that the proposals are in conformance with the Town of Milton's Engineering and Parks Standards Manual and any existing Sub-watershed Impact Studies, if applicable.
- Evaluate various stormwater management practices that meet the requirements of the Town and recommend a preferred preliminary strategy.
- Prepare a Stormwater Management (SWM) report documenting the preliminary strategy along with the technical information necessary for the justification and preliminary sizing of the proposed stormwater management facilities.



@2023 Google - Map Canada

CLIENT	MATTAMY HOMES CANADA
TITLE	FRAMGARD NORTH AND SOUTH BLOCKS SITE LOCATION

wsp

100 Commerce Valley Dr. West, Thornhill, ON Canada L3T 0A1
t: 905.882.1100 f: 905.882.0055 wsp.com

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Date JAN 2024	Proj. No. 231–00962–00	
Scale N.T.S.	Figure No. 1	Gr.No. 00

FIGURE 1-FRAMGARD.dwg Fig 1 C:\Users\CAAM078357\DC\ACCDocs\WSP Canada projects (AMER)\LDO\Files\231-00962-00 Framgard North and South Blocks\SWM\FIGURES\ Jan 19, 2024 - 10:07am

1.4 Design Criteria

The Town of Milton issued its “Engineering and Parks Standards Manual” in March 2019 and Conservation Halton issued its “Guidelines for Stormwater Management Engineering Submissions” in November 2021 to provide direction on the management of rainfall and runoff inside their agencies’ jurisdictions.

A Sub-watershed Impact Study (SIS) named the Boyne Survey Block 2 Sub-watershed Impact Study was issued in August 2016 by MTE Consultants Inc. The SIS provides stormwater management criteria specifically for Block 2 of the Community of Boyne in which the site is located within. Excerpts of the SIS are included in **Appendix A** of this report.

A summary of the stormwater management criteria applicable to this project follows:

1.4.1 Erosion Control

To minimize the impacts of erosion from the development, the flows generated by the 25mm 4-hour Chicago Storm must not exceed the Extended Detention/Erosion Control allowable discharge as stated in the SIS criteria established for the site. Refer to Section 1.4.4 below for more information on how the erosion control criteria will be addressed.

1.4.2 Water Quality

Under the guidelines and SIS criteria, the site is required to target a long-term removal of 80% of total suspended solids (TSS) on an annual loading basis.

1.4.3 Water Balance

A Water Balance Assessment report prepared by McClymont & Rak Engineers Inc. (MCR), dated January 2024, revealed a reduction of 1,177 m^3 of infiltration from the pre- to post-development conditions. The Town of Milton requires the site to retain stormwater on-site, to the extent practicable, to match the level of annual volume of overland runoff allowable from the development site under pre-development conditions. The North and South Blocks will be designed separately, to retain as much runoff as possible and address the infiltration volume deficiency through infiltration, evapotranspiration and reuse if appliable. Excerpts from the water balance assessment report can be found in **Appendix C**.

1.4.4 Water Quantity

The runoff generated from the Extended Detention/Erosion Control, 25-year, 100-year, and Regional design storms must not exceed the allowable release rates as stated in the SIS criteria established for the site. To meet the erosion control criteria, runoff generated from the 25mm 4-hour Chicago Storm event shall not exceed the Erosion Control/Extended Detention release rate. The North and South Blocks of the site will be designed to meet the water quantity requirements individually.

1.4.4.1 North Block

If stormwater is discharging to the existing SWM Pond “I” which is located north of the North Block, then the post-development release rate shall be attenuated to design flows that SWM Pond “I” was designed to receive from the site for all storms up to and including the 100-year and Regional storm events or the capacity of the existing pipe inlet to SWM Pond “I”, whichever is less.

1.4.4.2 South Block

If stormwater is to be discharged to the SWS-2-A channel directly, then the post-development release rate shall be attenuated to the unit allowable flow rates for the Extended Detention/Erosion Control, 25-year, 100-year, Regional storm events as established in the SIS for Boyne Block 2.

2 PRE-DEVELOPMENT CONDITIONS

2.1 General

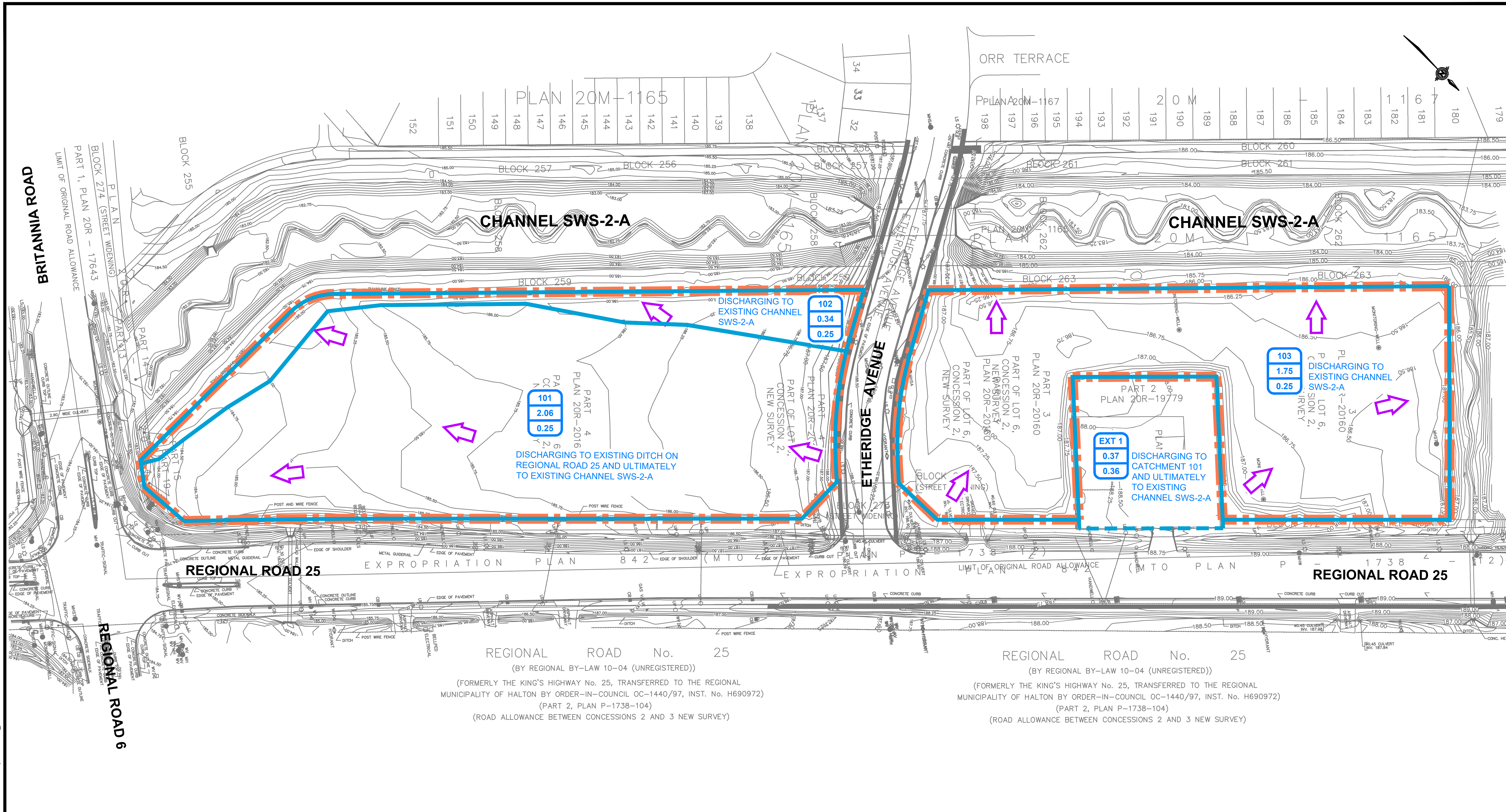
The existing site is presently a vacant lot comprising mainly of landscaping areas. The North Block of the site has a holdout property that will remain under post-development conditions which causes the site to have an irregular shape. However, the holdout area shall be included in the design of the North Block in the case of a future development.

The South Block has an area of 2.40 hectares and the drainage was determined based on the topographic survey conducted by Rady-Pentek & Edward Surveying Ltd. on February 5, 2018. The South Block is split into two catchments, **Catchments 101** and **102**, and are both primarily landscaping. **Catchment 101** has an area of 2.06 hectares and an estimated runoff coefficient of 0.25, the runoff from this catchment discharges directly to the ditch to the south and ultimately to the SWS-2-A channel. **Catchment 102** has an area of 0.34 hectares and an estimated runoff coefficient of 0.25 as well. The runoff from this catchment discharges directly to the SWS-2-A channel.

The North Block has an area of 1.75 hectares and the drainage was determined based on the topographic survey conducted by Rady-Pentek & Edward Surveying Ltd. on March 27, 2018. The North Block will be referred to as **Catchment 103**. The runoff from **Catchment 103** appears to ultimately discharge to the SWS-2-A channel. Runoff from the SWS-2-A channel is conveyed from north to south and ultimately discharges to Sixteen Mile Creek.

The North Block consists of primarily landscaping and has an estimated runoff coefficient of 0.25. The existing property on the North Block is an external area, with an area of 0.36 hectare and an estimated runoff coefficient of 0.36. The external area will be referred to as **Catchment EXT1** and its runoff drains through the site and discharges ultimately to the SWS-2-A channel. The external area consists of an existing single residential home and is predominately landscape area. This area is not currently owned by the client but will be considered as a future development and accounted for in the post-development conditions to meet the stormwater management criteria.

The existing condition of the site and drainage areas are shown in **Figure 2**.



LEGEND

- PROJECT BOUNDARY
- SUB-CATCHMENT BOUNDARY
- EXTERNAL SUB-CATCHMENT BOUNDARY
- 101 — SUB-CATCHMENT ID.
- 1.751 — DRAINAGE AREA (ha)
- 0.25 — AVERAGE RUNOFF COEFFICIENT
- DIRECTION OF SURFACE FLOW

0 10 20 30 40 50 100m

CLIENT

MATTAMY HOMES CANADA

TITLE

FRAMGARD NORTH AND SOUTH BLOCKS

EXISTING CONDITIONS

wsp
100 Commerce Valley Dr. West, Thornhill, ON Canada L3T 0A1
t: 905.882.1100 f: 905.882.0055 wsp.com

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Scale: AS SHOWN	Figure No.: 2 Gr.No.: -

FIGURE 2-FRAMGARD.dwg Fig 2 C:\Users\CAAM07~1\AppData\Local\Temp\AcPublish_21336\ Jan 19, 2024 - 10:13am

2.2 Rainfall Information

The rainfall intensity for the site was calculated using the following equation:

$$I = A(T_C + B)^C$$

Where:

- I = Rainfall intensity in mm/hr
- T_C = Time of concentration in hours
- A, B and C = Constant parameters stated in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual.

The parameters are summarized in **Table 2-1**.

Table 2-1: IDF Parameters used by The Town of Milton

Return Period (Years)	2	5	10	25	50	100
A	779	959	1089	1234	1323	1435
B	6	5.7	5.7	5.5	5.3	5.2
C	-0.8206	-0.8024	-0.7955	-0.7863	-0.7786	-0.7751

Source: The Town of Milton Engineering and Parks Standards (March 2019)

An initial time of concentration, T_c, of 10 minutes (or 0.167 hours) is recommended in the Town of Milton Engineering and Parks Standards Manual document as per Section 1.1.24.2.

2.3 Relevant Background Documents

The following background documents have been reviewed as part of the preparation of this report:

- Engineering and Parks Standards Manual, prepared by Town of Milton, dated March 2019.
- Conservation Halton Guidelines for Stormwater Management Engineering Submissions, prepared by Conservation Halton, dated November 2021.
- Boyne Survey Block 2 Subwatershed Impact Study, prepared by MTE Consultants Inc., dated August 25, 2016.

- Preliminary Geohydrology Assessment Northwestern Corner of Regional Road 25 and Britannia Road Milton, Ontario, prepared by McClymont & Rak Engineers Inc. (MCR), dated January 2024
- Geotechnical Report Residential Development Regional Road 25 and Britannia Road Milton, Ontario, prepared by McClymont & Rak Engineers Inc., dated July 2023
- Water Balance Assessment Report Northwest Corner of Regional Road 25 and Britannia Road Milton, Ontario prepared by McClymont & Rak Engineers Inc., dated January 2024
- Functional Servicing and Stormwater Management Report for Framgard South Major Node Mattamy (Milton West) Limited, prepared by David Schaeffer Engineering Ltd. (DSEL), dated March 2018.
- Functional Servicing and Stormwater Management Report for Framgard North Major Node Mattamy (Milton West) Limited, prepared by David Schaeffer Engineering Ltd., dated April 2018.
- Gulfbeck Developments Subdivision - Stormwater Management Design Report – SWM Pond I, prepared by The Municipal Infrastructure Group (TMIG) Ltd., dated September 2016.
- Mattamy Framgard Subdivision / Hydraulic Analysis of Tributary SWS-2-A, prepared by J.F. Sabourin and Associates Inc., dated Match 2018

The cover and relevant excerpts of the report prepared by MTE Consultants Inc., TMIG Ltd., and DSEL are included in **Appendix A**. The excerpts from the reports prepared by MCR are included in **Appendix C**.

2.4 Allowable Release Rates

As noted in Section 1.4.4, the allowable release rate for the North and South Blocks shall be established based on their discharge points. The South Block will discharge to the SWS-2-A channel while the North Block will discharge to the existing SWM Pond "I" located to the north of the site. The proposed area drains, catchbasins, storm sewers and storm piping on site shall be designed to capture and convey runoff from all storms up to and including the 100-year and the Regional storm events to the respective stormwater management facilities before discharging to the SWS-2-A channel.

2.4.1 North Block

Based on the Function Servicing and Stormwater Management Report for Framgard North Major Node Mattamy (Milton West) Limited, prepared by David Schaeffer Engineering Ltd., dated April 2018 and Stormwater Management Design Report – SWM Pond I, prepared by The Municipal Infrastructure Group (TMIG) Ltd., dated September 2016, the existing SWM Pond "I" was designed to receive the inflow from the North Block. The SWM Pond "I" was designed to receive an area of 2.13 ha with an imperviousness of 83% and a runoff coefficient of approximately 0.79.

There is also an inlet pipe constructed to receive the runoff from the North Block. The pipe is an 825 mm concrete pipe at 0.35%. The pipe was designed for the 5-year storm event for a catchment area of 2.13 ha and a runoff coefficient of 0.85. However, there is adequate capacity within the existing 825 mm concrete pipe and the downstream storm sewers to convey the 100-year and Regional storm event to SWM Pond "I".

Using the manning equation, the full flow pipe capacity of the inlet pipe is approximately 849.2 L/s. Therefore, the allowable release rate for the North Block is the release rate from the design area of 2.13 ha with a runoff coefficient of 0.79 or the capacity of the inlet pipe, whichever is less. The design flow rates are summarized below in **Table 2-3**. Please refer to **Appendix B** for the detailed calculations.

SWM Pond "I" is designed to provide the erosion control, water quality and, water quantity criteria for the North Block prior to discharging to the SWS-2-A channel. If the post-development release rates from the North Block is below the design flow rates for the North Block, the above-mentioned stormwater management criteria would be addressed for the North Block.

Table 2-2: Allowable Site Discharge Rate to SWM Pond "I" – North Block

Design Storm	Design Flow Rates Entering the SWM Pond "I" (L/s)*	Capacity of Inlet Pipe (L/s)	Allowable Flow Rate (L/s)
2	374.5	849.2	374.5
5	492.4		492.4
10	569.8		569.8
25	669.0		669.0
50	740.0		740.0
100	814.4		814.4

*Based on a catchment area of 2.13 ha, a runoff coefficient of 0.79 and a time of concentration of 10 minutes

2.4.2 South Block

Based on the SIS, discharges to the SWS-2-A channel are required to meet specific unit flow rates based on certain design storms. The South Block has a total area of 2.40 ha and the corresponding allowable release rates for the erosion control, 25-year, 100-year, and Regional storms are 1.4, L/s, 48 L/s and 120 L/s, and 168 L/s, respectively.

The allowable flow rates and the design unit flow rates are summarized below in **Table 2-4**. Please refer to **Appendix B** for the detailed calculations.

Table 2-3: Allowable Site Discharge Rate to SWS-2-A channel – South Block (Overall)

Design Storm	Unit Flow Rate (m^3/s/ha)*	Allowable Flow Rate (L/s)
Erosion Control (Extended Detention)	0.0006	1.4
25-year	0.020	48
100-year	0.050	120
Regional	0.070	168

*Note: From Table 4.2 of Section 4.3 of the Boyne Survey Block 2 Sub-watershed Impact Study (Aug 2016)

2.5 Groundwater and Dewatering System

Hydrogeological investigations were conducted by McClymont & Rak Engineers Inc. (MCR) in the Winter of 2023. A Geotechnical Report and Hydrogeological Investigation report was drafted and dated July 2023 and January 2024, respectively. The reports addressed the groundwater conditions, soil characterization and dewatering requirements. A Water Balance Assessment Report was also prepared by MCR and dated January 2024. The excerpts from the above reports can be found in **Appendix C**.

A total of 12 boreholes were drilled by another consultant, Shad & Associates Inc. in February - March 2018. While MCR drilled an additional nine boreholes in December 2022 – January 2023. Monitoring wells were also installed by Shad & Associates.

Based on the geotechnical investigation, the soil identified on-site included fill, silty sand / sandy silt, clayey silt / silty clay (till), sand, gravel / silty sand / sandy silt (till), and clayey silt till. The groundwater level ranged 0.74 m to 6.40 m before the existing ground surface.

The report indicates that the groundwater quality meets the Municipality of Halton Sewers By-Law criteria to discharge to the storm sewer network and sanitary sewer network. As a result, treatment is not required before the groundwater can be discharged to either municipal sewer system. It is proposed to discharge the groundwater to the municipal stormwater sewers during short-term / construction and in the long-term conditions. Groundwater is proposed to be discharged through the stormwater system and facilities before discharging to the SWS-2-A Channel.

The estimated long-term dewatering rates for the North block is 1.35 L/s and for the South Block is 1.09 L/s. These discharge rates will be accounted for in the SWM design for both North and South Blocks.

To address the water balance criteria and the infiltration volume deficit, infiltration trenches are proposed along the Natural Heritage System (N.H.S.) promenade located along the western limit of the development for both the South and North Blocks. Geotechnical investigation has been conducted near the N.H.S promenade. The design of the infiltration trenches will be based on the current available geotechnical information. It is recommended that additional investigations and tests are conducted at the location of the infiltration trenches to evaluate their feasibility and performance.

For the South Block, the four boreholes located closest to the proposed infiltration trenches are Boreholes 2, 4, 6, and 108. From the geotechnical investigation, it is shown that the highest groundwater level is 2.27 m below existing ground surface at an elevation of 182.83 m, measured at Borehole 4 on August 11, 2023. The soil in this area is expected to be composed of silty clay, clayey silt fill, clayey silt (till), clayey sandy silt (till), sandy till, and sandy silt (till).

For the North Block, the five boreholes located closest to the proposed infiltration trenches are Boreholes 8, 9, 10, 101, 103. From the geotechnical investigation, it is shown that the highest groundwater level is 2.58 m below existing ground surface at an elevation of 184.12 m, measured at Borehole 9 on April 6, 2023. The soil in this area is expected to be composed of silty clay, clayey silt fill, clayey silt (till), clayey sandy silt (till), silty sand/sandy silt, and sandy silt (till).

3 POST DEVELOPMENT CONDITIONS

3.1 General

The proposed development consists of mixed use buildings on both the North and South Block with a Natural Heritage System (N.H.S) promenade located along the western limit of both blocks. Please refer to the following sections and **Figure 3** for details of the post-development conditions, land uses, and stormwater catchments.

3.1.1 South Block

The proposed developments in the South Block will consist of a 15-storey building (Building 1), a 13-storey building (Building 2), a 11-storey building (Building 3),and a 14-storey building (Building 4) over two levels of underground parking. The developments will be surrounded by landscaping and two driveways, one from Regional Road 25 and one from Etheridge Avenue, that lead into the underground parking structure.

The South Block will be built in four separate phases with a new building per phase. As a result, WSP will be splitting the South Block into four catchments, **Catchments 201**, **202**, **203**, and **204**, respectively, where each one represents a phase of the development.

Two stormwater cisterns shall be proposed to control the flows from the South Block, with Cistern A, located in **Catchment 201**, designed to control flows from **Catchments 201** and **202** while Cistern B, located in **Catchment 203**, is designed to control flows from **Catchment 203** and **204**. The cisterns will attenuate the stormwater runoffs to the allowable release rate prior to discharging to SWS-2-A channel. Please refer to Section 3.4 for more details.

For the stormwater management strategy, it is assumed that **Catchments 201** to **204** will be constructed in sequence where **Catchment 201** will be constructed first followed by **202**, **203** and **204**, respectively.

It is currently assumed that there are no uncontrolled flows generated from the South Block. These assumptions will be confirmed at the detailed design stage.

An area breakdown of the overall South Block and per the two cisterns are provided below in **Tables 3-1** and **3-2**, respectively. Please refer to **Appendix B** for the detailed area breakdown for each catchment.

Table 3-1: Post-Development Area Breakdown – South Block (Overall)

Land Use	Area (m^2)	Runoff Coefficient, C	% Coverage
Impervious Roof Area	5,387	0.90	22
Soft Landscaping	3,821	0.25	16
At-Grade Impervious	14,795	0.90	62
Total	**24,003**	**0.80**	**100**

Table 3-2: Post-Development Area Breakdown – Per Cistern

Land Use	Area (m^2)	Runoff Coefficient, C	% Coverage
Cistern A (Catchments 201 & 202)			
Impervious Roof Area	3,047	0.90	26
Soft Landscaping*	1,286	0.25	11
At-Grade Impervious	7,563	0.90	64
Total	**11,895**	**0.83**	**100**
Cistern B (Catchments 203 & 204)			
Impervious Roof Area	2,340	0.90	19
Soft Landscaping	2,535	0.25	21
At-Grade Impervious	7,233	0.90	60
Total	**12,108**	**0.76**	**100**

3.1.2 North Block

The proposed development in the North Block will consist of a 15-storey building (Building 5), 12-storey building (Building 6), and a 14-storey building (Building 7) over two levels of underground parking. The developments will be surrounded by landscaping and driveways from both Regional Road 25 and Etheridge Avenue, that lead into the underground parking structure.

The North Block will be built in three separate phases with a new building per phase. As a result, WSP will be splitting the North Block into three catchments, **Catchments 205**, **206**, and **207**, respectively, where each one represents a phase of the development.

As previously mentioned, the existing holdout property, with an area of 0.36 ha will also drain into the site under post-development conditions. This area will be represented as **Catchment 208** in the post-development condition. Flows from this catchment will be captured within the North Block and be conveyed to SWM Pond “I”. This catchment was considered during the design of the existing SWM pond and will not add to the area the pond was initially designed for. Furthermore, it will be assumed that the holdout area will have a runoff coefficient of 0.85 for a conservative estimate of the total flows. This assumption is to be confirmed once a landscape plan for the holdout property is prepared.

In the post-development conditions, it is assumed that all four (4) catchments within the North Block and the holdout property will be conveyed to the SWM Pond “I”. These areas have a combined area of 2.12 ha and a runoff coefficient of 0.79.

For the stormwater management strategy, it is assumed that **Catchments 205** to **208** will be constructed in sequence where **Catchment 205** will be constructed first followed by **206**, **207** and **208**, respectively.

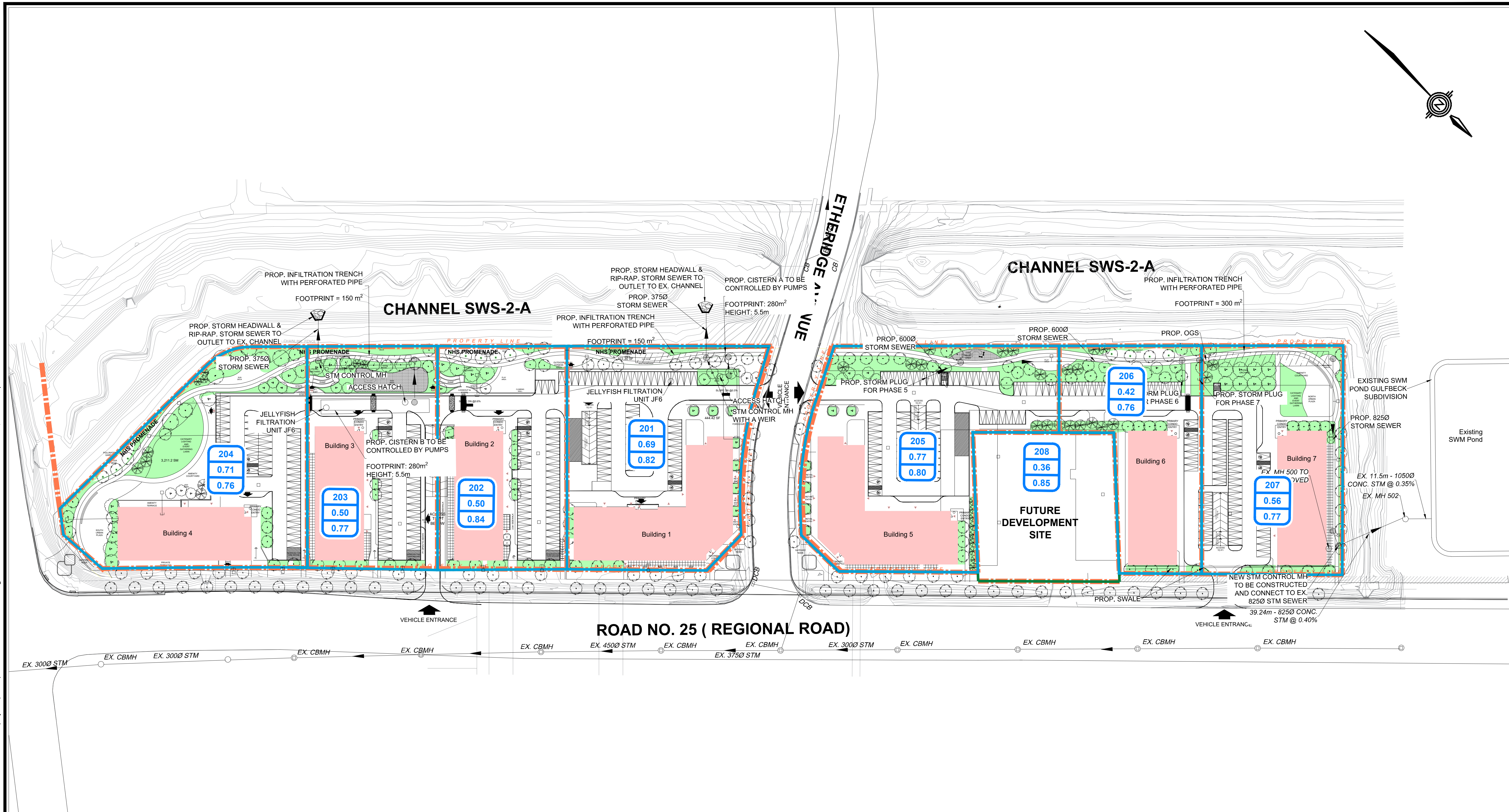
It is currently assumed that there are no uncontrolled flows generated from the North Block. These assumptions will be confirmed at the detailed design stage.

An area breakdown of the overall North Block is provided below in **Table 3-3**. Please refer to **Appendix B** for the detailed area breakdown for each catchment.

Table 3-3: Post-Development Area Breakdown – North Block (Overall)

Land Use	Area (m^2)	Runoff Coefficient, C	% Coverage
Impervious Roof Area	4,024	0.90	23
Soft Landscaping	3,202	0.25	18
At-Grade Impervious	10,282	0.90	59
Total	**17,508**	**0.78**	**100**
Phase 8 (Holdout Area)*	**3,648**	**0.85**	**-**
Grand Total	**21,156**	**0.79**	**-**

*Assumed 255 m^2 of landscape area in Phase 8



LEGEND

PROJECT BOUNDARY

SUB-CATCHMENT BOUNDARY

EXTERNAL BOUNDARY

IMPERVIOUS ROOF

PERVIOUS LANDSCAPE

201 — SUB-CATCHMENT ID.
2.36 — DRAINAGE AREA (ha)
0.25 — RUNOFF COEFFICIENT

CLIENT

MATTAMY HOMES CANADA

TITLE

FRAMGARD
NORTH AND SOUTH BLOCKS

PROPOSED CONDITIONS

wsp
100 Commerce Valley Dr. West, Thornhill, ON Canada L3T 0A1
t: 905.882.1100 f: 905.882.0055 wsp.com

Checked G.W.	Drawn AUTOCAD\A.M.
Date JAN 2024	Proj. No. 231-00962-00
Scale AS SHOWN	Figure No. 3

Gr.No. -

3.2 Erosion Control

As mentioned in Sections 1.4.1 and 1.4.4, the stormwater runoff generated from a 25mm 4-hour Chicago Storm must not exceed the Erosion Control/Extended Detention release rate limit. Analysis has been conducted to ensure that the SWM facilities utilized on each block has addressed this criterion and ensured compliance with the allowable released rates as per the SIS criteria established for the site. The North and South Blocks of the site are designed to meet the erosion control requirements individually. Please refer to Section 3.5 for more detailed information.

During construction the appropriate temporary Erosion and Sediment Controls shall be implemented to minimize construction impacts.

3.3 Water Quality Control

Stormwater runoff from the site will require water quality treatment. As stated in the Town's guideline, the standard for water quality treatment is the "Enhanced" level of protection which requires 80% TSS removal on a long-term average annual loading basis. This requirement is to be meet for both the North and South Blocks individually.

3.3.1 South Block

For the South Block, two filter units, both the Imbrium Jellyfish Filter JF6-5-1, are proposed to capture and treat all the at-grade areas of the South Block prior to discharging to the proposed stormwater cistern and ultimately to the SWS-2-A channel.

One unit will be used to treat runoff produced from **Catchments 201** and **202**, before being controlled by Cistern A located in **Catchment 201**. While the other unit will be used to treat runoff produced from **Catchments 203** and **204**, before being controlled by Cistern B located in **Catchment 203**. Both units will be located within the underground parking structure. Runoff from the proposed buildings rooftop is considered clean and is proposed to discharge to the proposed stormwater cisterns on site directly and to bypass the Jellyfish units.

The Jellyfish filter unit has a median TSS removal efficiency result of 89%, which is greater that the required 80%. The filter unit will provide the necessary water quality treatment for the South Block prior to discharging to the SWS-2-A channel. The proposed filter unit model is sized based on the most current landscape plan and will be confirmed in the detailed design stage. Jellyfish model sizing report and specifications are attached in **Appendix D**.

3.3.2 North Block

As per the Stormwater Management Design Report prepared by TMIG, dated September 2016, the existing SWM Pond “I” has been designed to provide water quality treatment that meets the required “Enhanced” level of protection prior to discharging to the SWS-2-A channel as long as the design release rates of the pond is met. Please refer to **Section 3.5** for more detailed information.

As best practice, an oil-grit separator (OGS), Imbrium Stormceptor EFO10, is proposed upstream of the North Block’s proposed infiltration trench to treat the stormwater prior to infiltrating storm runoff back into the native soil. The oil-grit separator will help capture hydrocarbons and sediments and improve the operations of the infiltration trench by reducing the chances that the hydrocarbons are infiltrated into the native soil and the perforated pipes are clogged. The oil-grit separator shall be constructed at the time that the construction of **Catchment 205** starts

The EFO sizing report and specifications are attached in **Appendix D**.

3.4 Water Balance

The proposed development will increase the site’s imperviousness by converting pervious surfaces to impervious surfaces, resulting in less infiltration and evapotranspiration and more runoff from the site. As noted in **Section 1.4.3** the target is to match post-development water balance to pre-development as much as possible as the regulating agencies stormwater management criteria. It is important to note that based on the Ontario’s Ministry of the Environment, Conservation and Parks, both the North and South Blocks are located in an area that is not considered to be a “Significant Groundwater Recharge Area” (SGRA) and does not have a “Highly Vulnerable Aquifer”.

The geotechnical consultant for the site, McClymont & Rak Engineers Inc. (MCR), prepared a Water Balance Assessment report dated January 2024 for the overall development analysing the North and South Blocks as one development. The report mentioned that the site soil is considered Class C, which has moderate infiltration potential. Under pre-development condition, approximately 93% of the site is considered permeable areas. While under post-development condition, approximately 55% of the site will be considered as permeable areas due to the proposed buildings, at-grade parking, walkways and driveways of the development. The post-development water balance compared to the pre-development water balance, identifies an expected reduction of 1,177 m^3 of infiltration volume.

WSP will be conducting a similar analysis to MCR and proposing a solution to address the infiltration deficit for each blocks individually. The stormwater management strategy will involve splitting the North and South Blocks into their separate group of catchments.

A conservative design infiltration rate of 5 mm/hr will be assumed for the design of the infiltration facilities. In-situ infiltration tests and additional soil investigation and groundwater monitoring at the location of the infiltration trenches is recommended during the detailed design stage to confirm the in-situ infiltration rates and the groundwater levels at the facilities.

To be conservative, all landscape areas located on top of the underground parking structure will be considered as impervious areas for the water balance analysis.

Additionally, it is important to note that all proposed infiltration facilities shall be located at least 5 m from any building foundations as per MCR report.

3.4.1 South Block

The South Block has a total area of 2.4 ha. Under pre-development condition, 100 % of the site is considered pervious areas while under post-development condition, 13.9 % of the site is considered pervious areas. As a result, there is an infiltration deficit of approximately 1,467 m^3/year. Please refer to **Table 3-4** for the water balance analysis.

Table 3-4: Water Balance Analysis – South Block (No SWM Facilities)

Water Balance	Pre-Development		Post-Development		Differences (m^3/year)
	Volume (m^3/year)	%	Volume (m^3/year)	%	
Reuse	0	0.0%	0	0.0%	0
Infiltration	1,704	8.8%	237	1.2%	-1,467
Evapotranspiration	14,474	74.7%	3,680	19.0%	-10,794
Runoff	3,192	16.5%	15,453	79.8%	12,261
Precipitation (Total)	19,371	100%	19,371	100%	-

To address the infiltration deficit, two (2) infiltration trenches are proposed within the **Catchments 201** and **203**. The infiltration trenches shall be designed to infiltrate a 1 mm rainfall volume within 48 hours at a minimum. Since the proposed underground parking structure spans much of the site, the infiltration trenches will be located along the western property limit of the site underneath the N.H.S. promenade.

Runoff from the site will be treated by the respectively proposed filter units prior to discharging to the respective stormwater cistern located within the underground parking structure. Each stormwater cistern and its pump will attenuate the flows to the allowable release rates where it will discharge the runoff to a storm sewer network which will convey the flow to the proposed outfall and out to the SWS-2-A channel.

With the flows attenuated to a lower release rate and cleaned by the filter unit, an infiltration trench is proposed as part of each storm sewer network. A segment of the storm sewer network will consist of perforated pipes wrapped in geotextile fabric located on top of 0.60 m clearstone layer wrapped in geotextile fabric. As water passes through the perforated pipe, runoff will seep in to the clearstone layer and infiltrate into the native soil. The footprint of the infiltration trench is designed to be 150 m^2. Assuming a height of 0.60 m and a porosity of 0.40, the infiltration trench will have a volume of 36 m^3.

As there are two outlets and **Catchments 201** and **202** (1.19 ha) are similar to **Catchments 203** and **204** (1.21 ha), two infiltration trenches with an area of 150 m^2 and a volume of 36 m^3 is proposed for both outlets. Additional details will be provided at the detailed design stage. However, the design and infiltration capacity of the infiltration trenches shall be confirmed at the detailed design stage once additional groundwater monitoring and in-situ infiltration tests are conducted.

Table 3-5 summarizes the water balance analysis for the South Block with the proposed infiltration trenches. Detailed water balance calculations are included in **Appendix B** of this report and the Water Balance Report prepared by MCR can be found in **Appendix C**. Please refer to **Figure 3** for the location of the infiltration trench.

Table 3-5: Water Balance Analysis – South Block (With SWM Facilities)

Water Balance	Pre-Development		Post-Development		Differences (m^3/year)
	Volume (m^3/year)	%	Volume (m^3/year)	%	
Reuse	0	0.0%	0	0.0%	0
Infiltration	1,704	8.8%	1,738	9.0%	34
Evapotranspiration	14,474	74.7%	3,680	19.0%	-10,794
Runoff	3,192	16.5%	13,952	72.0%	10,760
Precipitation (Total)	19,371	100%	19,371	100%	-

Should infiltration not be feasible, a water reuse sump volume within the proposed stormwater cisterns will be designed to store the equivalent of the 1 mm storm event. Thus, ensuring the water balance requirement for the South Block can be addressed.

Water stored within the sump can be used for irrigation supply for the landscaped area on-site and/or greywater demands for toilet flushing in the communal areas of the development. More details will be provided at the detailed design stage if required. Coordination will be required with the mechanical, landscape and irrigation consultant to confirm the demands and reuse system.

The mechanical design of the rainwater reuse pump system from the cistern will ensure that the cistern is empty prior to switching to the municipal water supply. It is important to note that only the required water balance volume will have to be reused within 72 hours after the start of the rainfall event to meet the water balance criteria.

3.4.2 North Block

The North Block, including the holdout property, has a total area of 2.12 ha. Under pre-development condition, 97.1 % of the site is considered pervious areas while under post-development condition, 13.1 % of the site is considered pervious areas. As a result, there is an infiltration deficit of approximately 1,262 m^3/year. Please refer to **Table 3-6** for the water balance analysis.

Table 3-6: Water Balance Analysis – North Block (No SWM Facilities)

Water Balance	Pre-Development		Post-Development		Differences (m^3/year)
	Volume (m^3/year)	%	Volume (m^3/year)	%	
Reuse	0	0.0%	0	0.0%	0
Infiltration	1,459	8.5%	197	1.2%	-1,262
Evapotranspiration	12,490	73.2%	3,159	18.5%	-9,330
Runoff	3,124	18.3%	13,716	80.3%	10,592
Precipitation (Total)	17,073	100%	17,073	100%	-

To address the infiltration deficit, one large infiltration trench is proposed within the **Catchment 207**. The infiltration trench shall be designed to infiltrate a 1 mm rainfall volume within 48 hours at a minimum. Since the proposed underground parking structure spans much of the site, the infiltration trenches will be located along the western property limit of the site underneath the N.H.S. promenade.

Runoff from the site will convey to a proposed storm sewer located within the N.H.S. promenade and the north limit of **Catchment 207** to the existing storm sewer connecting the site the SWM Pond "I" to the north and ultimately to the SWS-2-A channel. An oil-grit separator is proposed upstream of the infiltration trench to treat the runoff prior to infiltrating into the native soil.

With the flows cleaned by the oil-grit separator, an infiltration trench is proposed as part of the storm sewer network. A segment of the storm sewer network will consist of perforated pipes wrapped in geotextile fabric located on top of 0.60 m clearstone layer wrapped in geotextile fabric. As water passed through the perforated pipes, runoff will seep in to the clearstone layer and infiltrate into the native soil. The footprint of the infiltration trench is designed to be 300 m^2. Assuming a height of 0.60 m and a porosity of 0.40, the infiltration trench will have a volume of 72 m^3. Additional details will be provided at the detailed design stage. However, the design and infiltration capacity of the infiltration trenches shall be confirmed at the detailed design stage once additional groundwater monitoring and in-situ infiltration tests are conducted. The storm sewer network, oil-grit separator and infiltration trench shall be constructed at the time that the construction of **Catchment 205** starts.

Table 3-7 summarizes the water balance analysis for the North Block with the proposed infiltration trench. Detailed water balance calculations are included in **Appendix B** of this report and the Water Balance Report prepared by MCR can be found in **Appendix C**. Please refer to **Figure 3** for the location of the infiltration trench.

Table 3-7: Water Balance Analysis – South Block (With SWM Facilities)

Water Balance	Pre-Development		Post-Development		Differences (m^3/year)
	Volume (m^3/year)	%	Volume (m^3/year)	%	
Reuse	0	0.0%	0	0.0%	0
Infiltration	1,459	8.5%	1,680	9.8%	221
Evapotranspiration	12,490	73.2%	3,159	18.5%	-9,330
Runoff	3,124	18.3%	12,233	71.7%	9,109
Precipitation (Total)	17,073	100%	17,073	100%	-

Should infiltration not be feasible, a water reuse sump volume within the proposed stormwater cisterns will be designed to store the equivalent of the 1 mm storm event. Thus, ensuring the water balance requirement for the South Block can be addressed.

Water stored within the sump can be used for irrigation supply for the landscaped area on-site and/or greywater demands for toilet flushing in the communal areas of the development. More details will be provided at the detailed design stage if required. Coordination will be required with the mechanical, landscape and irrigation consultant to confirm the demands and reuse system.

The mechanical design of the rainwater reuse pump system from the cistern will ensure that the cistern is empty prior to switching to the municipal water supply. It is important to note that only the required water balance volume will have to be reused within 72 hours after the start of the rainfall event to meet the water balance criteria.

3.5 Water Quantity Control

As stated in **Section 1.4.4** and **2.4**, the post-development flows from the North and South Blocks and any external area that will be captured by the proposed development will be attenuated to the allowable release rates to either SWM Pond “I” located on the north side of the development, or the SWS-2-A channel located along the west side of the development.

Please note that all roof drains, catchbasins and area drains for both the North and South Blocks are to be designed by the mechanical consultants and shall capture and convey the runoff from the site and external area up to the 100-year and Regional storm event, whichever is greater.

3.5.1 South Block

The entire South Block will discharge to the SWS-2-A channel. Unit Flow rates have been established for the discharge to the SWS-2-A channel for the Extended Detetion/Erosion Control, 25-year, 100-year and Regional storm events. It is currently assumed that there are no uncontrolled flows generated from the South Block. These assumptions will be confirmed at the detailed design stage.

Two outlets are proposed for the South Block, **Catchments 201** and **202** will discharge to a proposed north outfall to the SWS-2-A channel located west of **Catchment 201**. While **Catchments 203** and **204** will discharge to a proposed south outfall to the SWS-2-A channel located west of **Catchment 203**. Each outlet shall have its own stormwater management facilities to ensure that the post-development release rates are attenuated to the allowable release rates.

It was confirmed with Conservation Halton that the floodplain hazard is contained within the existing SWS-2-A channel and should not impact the site. Based on the Hydraulic Analysis of Tributary SWS-2-A memo prepared by J.F. Sabourin and Associates Inc. dated March 16, 2018, the water elevation in the creek during the Regional storm event near the north and south outfalls are approximately 184.13 m and 183.99 m, respectively. These elevations are from HEC-RAS River Stations 85 and 68, respectively. The outfall will be designed by the geomorphologists consultant prepared under a separate cover and coordinated with WSP at the detailed design stage.

3.5.1.1 Catchments 201 and 202

Catchments 201 and **202** has a total area of 1.19 ha and an imperviousness of 89%. Based on the unit flow rates mentioned in Section 2.4, the allowable release rates for **Catchments 201** and **202** are summarized in **Table 3-8** below.

Table 3-8: Allowable Site Discharge Rate to SWS-2-A channel – South Block (Catchments 201 and 202)

Design Storm	Unit Flow Rate ($m^3/s/ha$)*	Allowable Flow Rate (L/s)
Erosion Control (Extended Detention)	0.0006	0.71
25-year	0.020	23.8
100-year	0.050	59.5
Regional	0.070	83.3

*Note: From Table 4.2 of Section 4.3 of the Boyne Survey Block 2 Sub-watershed Impact Study (Aug 2016)

A stormwater cistern, referred to as Cistern A, will be proposed within the underground parking structure in **Catchment 201** and is designed to receive the runoff from **Catchments 201** and **202** for all storms up to and including the 100-year and Regional storm event.

As mentioned in Section 2.5, the long-term dewatering rate of the South Block is approximately 1.09 L/s. It is proposed to discharge the long-term dewatering to the development storm sewer system and ultimately to the SWS-2-A channel. It is assumed that the groundwater dewatering rate is split evenly between the two cisterns, with a constant flow of 0.55 L/s discharging to Cistern A.

Using a Visual OTTHYMO (VO6) model for the project and the 4-hr Chicago storm distribution, the storage volume of the cistern was determined iteratively. The model was used to calculate the discharge rates achieved by the proposed flow controls under all storm events using the 4-hr Chicago storm distribution derived from the Town of Milton IDF curves.

Cistern A is designed to have a footprint of 280 m^2 with a height of 5.5 m, providing a total volume of 1,540 m^3 for quantity control. Pump(s) with a maximum release rate of 83.3 L/s are proposed to pump the stormwater at the controlled release rates for the respective storm events. The design of the pump will be provided for the mechanical consultant for the project. More details will be provided at the detailed design stage.

The flows will be pumped to a higher elevation than the base of the cistern to a storm manhole located within the underground parking structure. Runoff from the storm manhole shall flow by gravity to the SWS-2-A channel via a proposed storm sewer network and outfall. As mentioned in Section 3.4, a section of the storm sewer network will consist of perforated pipes wrapped in geotextile fabric on top of a clearstone layer also wrapped in geotextile fabric. This set up will create an infiltration trench allowing stormwater to infiltrate into native soil before discharging to the SWS-2-A channel and addressing the water balance criteria.

In the situation where a storm that exceeds the 100-year/Regional storm event occurs or the outlet pipe is blocked; an emergency overflow will be provided in the cistern via an access hatch and the manhole. Excess stormwater will be discharged to grade and flow to the SWS-2-A channel directly via overland flow. Backflow preventors will also be proposed at the inlet pipes to the cistern. This will prevent flow from backing up into the building pipework.

A summary of the modelling results is provided below in **Table 3-9**. Note that the simulated situation includes an empty sump at the beginning of each rainfall event. Full VO6 modelling output is provided in **Appendix E**.

Table 3-9: Summary of Modelling Results – South Block (Catchments 201 and 202)

Return Period	Utilized Cistern Storage (m^3/1540 m^3)	Post-Development Release Rate (L/s)	Allowable Release Rate (L/s)
Erosion Control (Extended Detention)	259	0.71	0.71
25-year	624	14.0	23.8
100-year	726	23.0	59.5
Regional	1,442	75.0	83.3

The modelling results demonstrates that the Regional storm event uses a maximum storage volume of 1,442 m^3 within the cistern, which is below the storage volume provided. The overall peak flow rates for all four storm events are less than the allowable release rate. Therefore, the water quantity requirement is addressed for **Catchments 201** and **202**.

3.5.1.2 Catchments 203 and 204

Catchments 203 and **204** has a total area of 1.21 ha and an imperviousness of 79%. Based on the unit flow rates mentioned in Section 2.4, the allowable release rates for **Catchments 203** and **204** are summarized in **Table 3-10** below.

Table 3-10: Allowable Site Discharge Rate to SWS-2-A channel – South Block (Catchments 203 and 204)

Design Storm	Unit Flow Rate (m^3/s/ha)*	Allowable Flow Rate (L/s)
Erosion Control (Extended Detention)	0.0006	0.73
25-year	0.020	24.2
100-year	0.050	60.5
Regional	0.070	84.8

*Note: From Table 4.2 of Section 4.3 of the Boyne Survey Block 2 Sub-watershed Impact Study (Aug 2016)

A stormwater cistern, referred to as Cistern B, will be proposed within the underground parking structure in **Catchment 203** and is designed to receive the runoff from **Catchments 203** and **204** for all storms up to and including the 100-year and Regional storm event.

As mentioned in Section 2.5, the long-term dewatering rate of the South Block is approximately 1.09 L/s. It is proposed to discharge the long-term dewatering to the development storm sewer system and ultimately to the SWS-2-A channel. It is assumed that the groundwater dewatering rate is split evenly between the two cisterns, with a constant flow of 0.55 L/s discharging to Cistern B.

Using a Visual OTTHYMO (VO6) model for the project and the 4-hr Chicago storm distribution, the storage volume of the cistern was determined iteratively. The model was used to calculate the discharge rates achieved by the proposed flow controls under all storm events using the 4-hr Chicago storm distribution derived from the Town of Milton IDF curves.

Cistern B is designed to have a footprint of 280 m^2 with a height of 5.5 m, providing a total volume of 1,540 m^3 for quantity control. Pump(s) with a maximum release rate of 84.8 L/s are proposed to pump the stormwater at the controlled release rates for the respective storm events. The design of the pump will be provided for the mechanical consultant for the project. More details will be provided at the detailed design stage.

The flows will be pumped to a higher elevation than the base of the cistern to a storm manhole located within the underground parking structure. Runoff from the storm manhole shall flow by gravity to the SWS-2-A channel via a proposed storm sewer network and outfall. As mentioned in Section 3.4, a section of the storm sewer network will consist of perforated pipes wrapped in geotextile fabric on top of a clearstone layer also wrapped in geotextile fabric. This set up will create an infiltration trench allowing stormwater to infiltrate into native soil before discharging to the SWS-2-A channel and addressing the water balance criteria.

In the situation where a storm that exceeds the 100-year/Regional storm event occurs or the outlet pipe is blocked; an emergency overflow will be provided in the cistern via an access hatch and the manhole. Excess stormwater will be discharged to grade and flow to the SWS-2-A channel directly via overland flow. Backflow preventors will also be proposed at the inlet pipes to the cistern. This will prevent flow from backing up into the building pipework.

A summary of the modelling results is provided below in **Table 3-11**. Note that the simulated situation includes an empty sump at the beginning of each rainfall event. Full VO6 modelling output is provided in **Appendix E**.

Table 3-11: Summary of Modelling Results – South Block (Catchments 203 and 204)

Return Period	Utilized Cistern Storage (m^3/1540 m^3)	Post-Development Release Rate (L/s)	Allowable Release Rate (L/s)
Erosion Control (Extended Detention)	245	0.73	0.73
25-year	612	13.0	24.2
100-year	712	23.0	60.5
Regional	1,451	73.0	84.8

The modelling results demonstrates that the Regional storm event uses a maximum storage volume of 1,451 m^3 within the cistern, which is below the storage volume provided. The overall peak flow rates for all four storm events are less than the allowable release rate. Therefore, the water quantity requirement is addressed for **Catchments 203** and **204**.

3.5.2 North Block

As mentioned previously, runoff from the North Block including the holdout property will discharge to the existing SWM Pond "I" located north of the development. The SWM Pond "I" was designed to receive the storm runoff from the North Block for all storms up to and including the 100-year and Regional storm event.

A storm sewer will be built along the N.H.S. Promenade and the north side of **Catchment 207** to convey the runoff from the entire site to the existing 825 mm storm sewer connecting the site to the pond for all storm events up to and including the 100-year and Regional storm event. It is currently assumed that there are no uncontrolled flows generated from the North Block. These assumptions will be confirmed at the detailed design stage. The storm sewer network, oil-grit separator and infiltration trench shall be constructed at the time that the construction of **Catchment 205** starts.

As long as the post-development flows from the site are less than the design flows entering the SWM Pond "I", the existing Pond will provide the necessary erosion control, water quality, and water quantity controls for the North Block before discharging to the SWS-2-A channel.

As mentioned in Section 2.4, SWM Pond "I" was designed assuming runoff from 2.13 ha with a runoff coefficient of 0.79 from the North Block will discharge to the facility. Under post-development condition, the total area of the North Block is 2.12 ha with a runoff coefficient of 0.79.

As mentioned in Section 2.5, the long-term dewatering rate of the North Block is approximately 1.35 L/s. It is proposed to discharge the long-term dewatering to the development storm sewer system to the SWM Pond “I”, and ultimately to the SWS-2-A channel.

As the post-development area is the less than the design area, no additional quantity control is required for the North Block. Using the modified rationale method with a time of concentration of 10 minutes, the flows from the North block are summarized in **Table 3-12**. It is assumed the steady state discharge rate of 1.35 L/s from the long-term dewatering of the North Block will discharge to the SWM Pond “I”.

Table 3-12: Summary of Post-Development Flows – North Block (Overall)

Return Period	Post-Development Release Rate (L/s)*	Allowable Release Rate (L/s)**
2	375.1	374.5
5	492.7	492.4
10	570.0	569.8
25	669.0	669.0
50	739.8	740.0
100	814.1	814.4

*A = 2.12 ha C = 0.79 with a time of concertation of 10 minutes. Including 1.0 L/s from the expected long-term dewatering of the North Block

**A = 2.13 ha, C = 0.79 with a time of concertation of 10 minutes.

As shown in **Table 3-12**, the flow is less than the allowable release rate for storms greater than the 25-year storm event. However, there is a slight exceedance of a maximum flow of 0.60 L/s during the 2-year storm event. The exceedances are deemed to be negligible to have a significant impact to a SWM Pond “I”, which is designed for a total catchment area of 34.2 ha. Additionally, the proposed infiltration trench in the North Block will take in some runoff volume from the development before discharging to the pond. Therefore, no additional quantity control is required.

It is important to note that as per “SWM Pond “I” Cross Section Details (1 of 2)” engineering drawing prepared by TMIG dated September 9, 2016, the spillway elevation of the pond is 186.50. The grading of the North Block shall be designed to ensure that the top of all storm structures is higher than 186.80, which represents the top of the emergency spillway elevation of SWM Pond “I”. This will ensure that during storms greater than the Regional storm event and in the instance where the pond’s outlet is blocked, excess water will spill through the pond emergency spillway and not through the North Block. A retaining wall will be required along the western limit of the North Block to prevent this condition.

Furthermore, WSP would like to note that in the SWM Pond “I” report prepared by TMIG dated September 2016, a storm sewer design sheet was included. The storm sewer design sheet was used to design the sewers to convey the 5-year storm events, but WSP checked if the storm sewers can convey the 100-year storm event as well.

The existing 825 mm storm sewer on site was designed for a catchment area of 2.13 ha with a runoff coefficient of 0.85. The 825 mm storm sewer conveys the runoff downstream to a 1050 mm storm sewer which also receives runoff from a 750 mm storm sewer. The existing 750 mm storm sewer was designed for a catchment area of 1.76 ha with a runoff coefficient of 0.90.

WSP checked that during the 100-year storm event for the 2.13 ha and 1.76 ha catchments, there is still capacity in the 750 mm, 825 mm and 1050mm pipe to convey the 100-year storm event to the pond. Therefore, no storm sewers upgrades are required as the 100-year peak flow is larger than the Regional Storm peak flow. Please refer to **Appendix A** for the documents from TMIG and **Appendix B** for the detailed calculations.

4 CONCLUSIONS

This stormwater management report has been prepared to support the Rezoning Application for the proposed Framgard North and South Blocks development in the Town of Milton. The key points are summarized below.

Erosion Control

For the South Block, the proposed SWM facilities shall control the stormwater discharge to the Extended Detention/Erosion Control limits established by the SIS criteria. While for the North Block, the Extended Detention/Erosion Control will be provided by the existing SWM Pond "I". Temporary erosion and sediment control measures will be implemented during construction.

Water Quality

For the South Block, two (2) Jellyfish Filters (JF6-5-1) are proposed to treat the at-grade runoff from the four drainage catchments of the South Block prior to discharging to the proposed cisterns and ultimately, the SWS-2-A Channel. Runoff from the rooftop is considered clean and will bypass the treatment units.

For the North Block, the existing SWM Pond "I" has been designed to provide water quality treatment to meet the required "Enhanced" level of protection. An OGS (EFO10) is proposed to provide pre-treatment for North Block prior to discharging to the proposed infiltration trench to improve the effectiveness and operations of the facility.

Water Balance

For both the North and South Blocks, infiltration trenches are proposed along the western limit of the site to infiltrate runoff from the Blocks to the native soil to reduce the infiltration deficit created because of the proposed development. The feasibility of the infiltration trench will depend on additional geotechnical investigation at the location of the proposed infiltration trench. If infiltration is not feasible, water reuse systems will be proposed for the blocks.

Water Quantity

The South Block will consist of two outlets to the existing channel SWS-2-A. **Catchments 201** and **202**, representing Phases 1 and 2, will discharge to the north outlet while **Catchments 203** and **204**, representing Phases 3 and 4 will discharge to the south outlet. To meet the allowable release rates, a cistern with a footprint of approximately 280 m^2 and a height of 5.5 m is proposed for each outlet. The cisterns will be located within the underground parking structure of **Catchments 201** and **203**. Pumps are proposed with a maximum pump rate of 83-85 L/s to discharge to the proposed storm sewers within the N.H.S. Promenade before outletting to the SWS-2-A channel.

For the North Block, flows from the post-development conditions is less than the design flows that the SWM Pond “I” was designed to receive. Therefore, no additional quantity control is required. A storm sewer is proposed along the N.H.S. Promenade and the north side of **Catchment 207** to convey the runoff from the North Block to the existing storm sewers on site which will discharge the runoff to SWM Pond “I”. SWM Pond “I” will provide the necessary erosion control, water quality and water quantity for the North Block before the runoff is discharged to the SWS-2-A channel.

This report has demonstrated that the proposed SWM strategies will address the stormwater management related impacts from this project and meet the intent of the Town of Milton and Conservation Halton guidelines and the criteria established in the Boyne Survey Block 2 SIS. Based on the proposed SWM facilities and the requirements stated in the Engineering and Parks Standards Manual, it is deemed the Town’s stormwater infrastructure can support the proposed development.

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APPENDIX

A

Background Documents

A-1 Excerpts from MTE SIS Report (Aug 2016)

BOYNE SURVEY BLOCK 2

FINAL

Subwatershed Impact Study

Project Location:
Town of Milton

Prepared for:
Block 2
Land Owners Group

Prepared by:
MTE Consultants Inc.
1016 Sutton Drive, Unit A
Burlington, ON L7L 6B8

Draft: June 28, 2013
Revised: March 28, 2014
Revised: July 3, 2015
Final: August 25, 2016

MTE File No.: 10477-100

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Notwithstanding the above, a private well monitoring program should be implemented for due diligence purposes to include private wells within close proximity to the Site. This due diligence monitoring will provide adjacent land owners with the monitoring data to demonstrate that groundwater conditions have not been significantly affected by the development. The monitoring program should include the collection of groundwater samples to document baseline chemistry and bacteria presence/absence (pre-development, construction, post-development); and the measurement of water levels on a quarterly basis (pre-development, construction, post development). This is further discussed in Section 6.1 of the report

It should be noted that the on-site monitoring wells must be decommissioned in accordance with Ontario Regulation 903 at such time that the monitoring wells are no longer needed for monitoring.

4.0 STORMWATER MANAGEMENT

4.1 Existing Drainage Conditions

A review of the existing drainage conditions within the area was carried out using the available topographic mapping, air photography, site inspection and information provided by others. Topographical information provided by the Boyne Survey Landowners Group, which was generated using LIDAR mapping techniques, forms the basis of the existing drainage conditions assessment.

As shown in Figure 4.1, the majority of the Study Area (i.e. Block 2) lies within the limits of Subwatershed No. 2 of the Sixteen Mile Creek Watershed. The balance of Block 2, lying immediately east of Bronte Street South, is within the Indian Creek watershed.

The existing drainage characteristics of the Study Area were previously analyzed within the Final Draft Sixteen Mile Creek Areas 2 and 7 Subwatershed Update Study (SUS; AMEC Environment and Infrastructure, May 2015). Appendix C1 contains a figure from that report (Drawing No. 5) illustrating drainage conditions assumed in that report.

The existing drainage conditions for the Study Area presented in the SUS were verified using the more detailed topographical information and are illustrated on Figure 4.2. The upstream contributing drainage areas external to the Study Area are shown on Figure 4.3. The drainage areas for the external contributing areas were derived from the drainage area plans for the subdivisions north of Louis St. Laurent Avenue. Two watercourses, referred to as SWS-1-A and SWS-2-A, traverse the Study Area from north to south. Referring to Figure 4.4, (Drainage patterns south of Britannia Road), these two features confluence approximately 500 m south of Britannia Road. The combined feature then drains in a southeast direction across Regional Road 25, through the Rattlesnake Point Golf Club, and towards the main branch of the Sixteen Mile Creek. The combined feature confluences with the main branch of Sixteen Mile Creek just north of Lower Baseline Road.

Figure 4.5 illustrates refined existing conditions drainage catchments in the vicinity of the woodland and wetland features located directly east of Regional Road 25. Catchment 107C includes the total area that drains through the woodland to the wetland feature (approximately 3.3 ha surrounding and including the woodland). The area within Catchment 107B, which includes approximately 4.5 ha north of the woodland and wetland areas, drains to the western edge of the wetland feature. Combined flows from this small feature are conveyed within a short unregulated drainage feature which is referred to as SWS-2-A-1. Catchment 107A drains directly to Regional Road 25. All three areas confluence at the culvert crossing of Regional Road 25 approximately 540m north of Britannia Road, which conveys drainage to the middle portion of SWS-2-A.

The Study Area is comprised mainly of undeveloped tableland. There are several existing residences and associated agricultural buildings fronting onto Bronte St. S. and Regional Road 25.

Project: 10477-100 CAD: P:\P\10477\LD\10477-100-MS3-SUBWATERSHED IMPACT AREAS.DWG

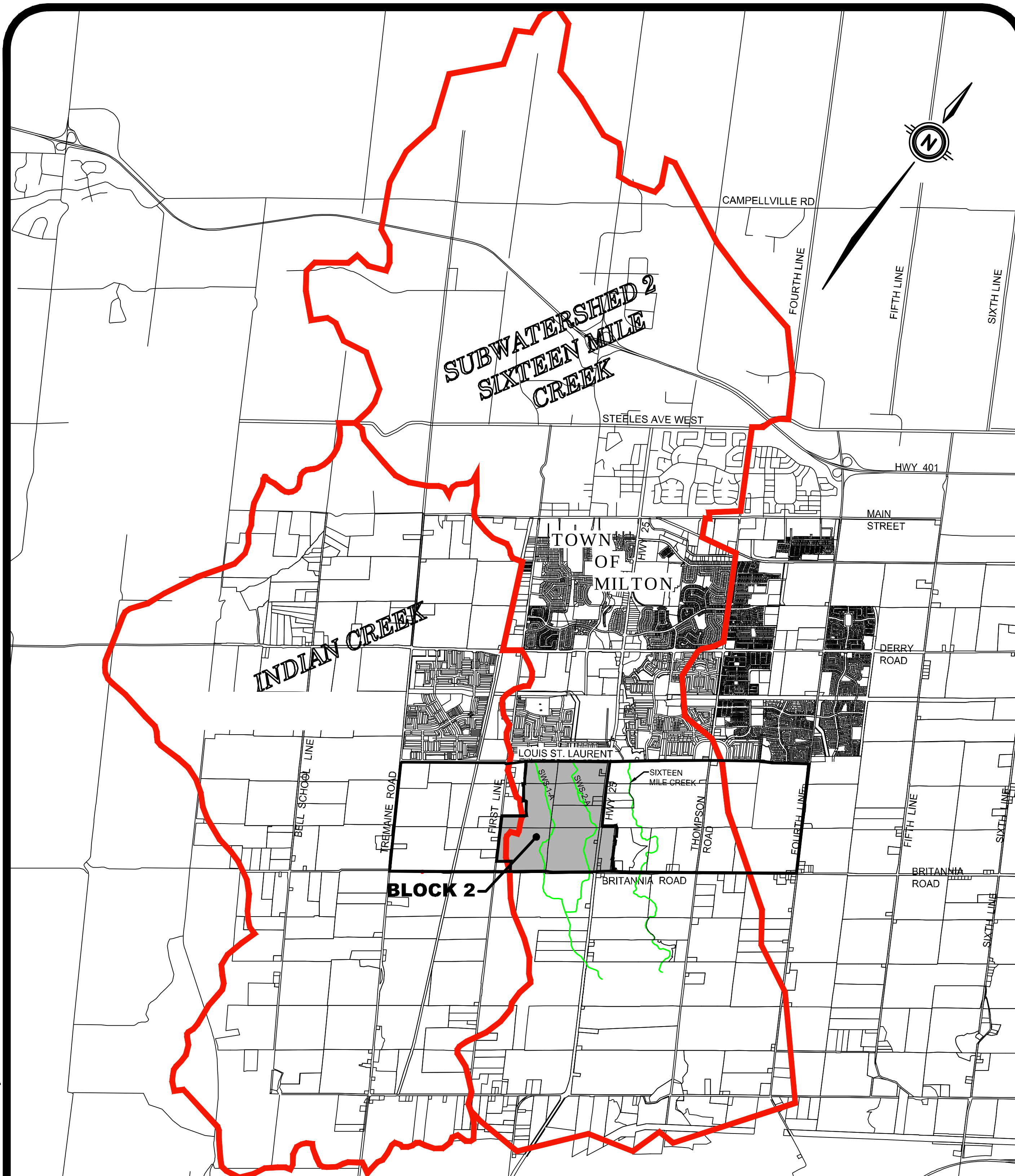


FIGURE 4.1 - SIXTEEN MILE CREEK SUBWATERSHED LIMITS

MTE

Engineers | Scientists | Surveyors

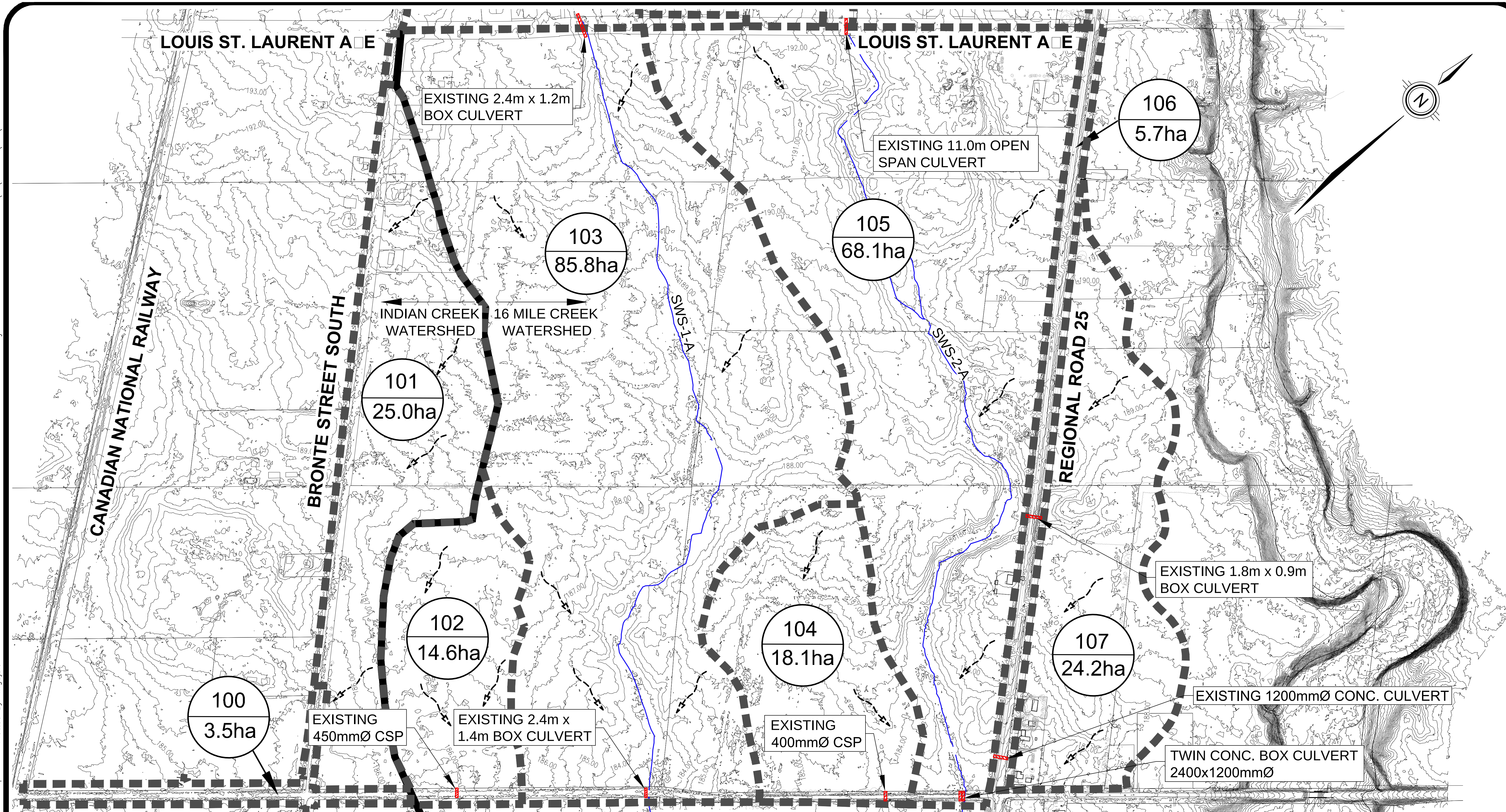
Project Name	
SUBWATERSHED IMPACT STUDY - BLOCK 2	
Site	**Client**
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale.	MTE Project No.	Date	Layout No.
1:75,000	10477-100	JUNE 2015	FIG 4.1

June 25, 2015 — 1:41 p.m. — Plotted By: rmoir

Project: 10477-100 CAD: P:\P\10477\LD\10477-100-ST1.DWG

July 2, 2015 — 1:22 p.m. — Plotted By: mgojsic



LEGEND

101	CATCHMENT ID
22.3ha	CATCHMENT AREA IN HECTARES
	CATCHMENT LIMIT
	EXISTING WATERSHED BOUNDARY
	EXISTING DIRECTION OF DRAINAGE

FIGURE 4.2 - EX. DRAINAGE CONDITIONS WITHIN STUDY AREA

MTE
Engineers | Scientists | Surveyors

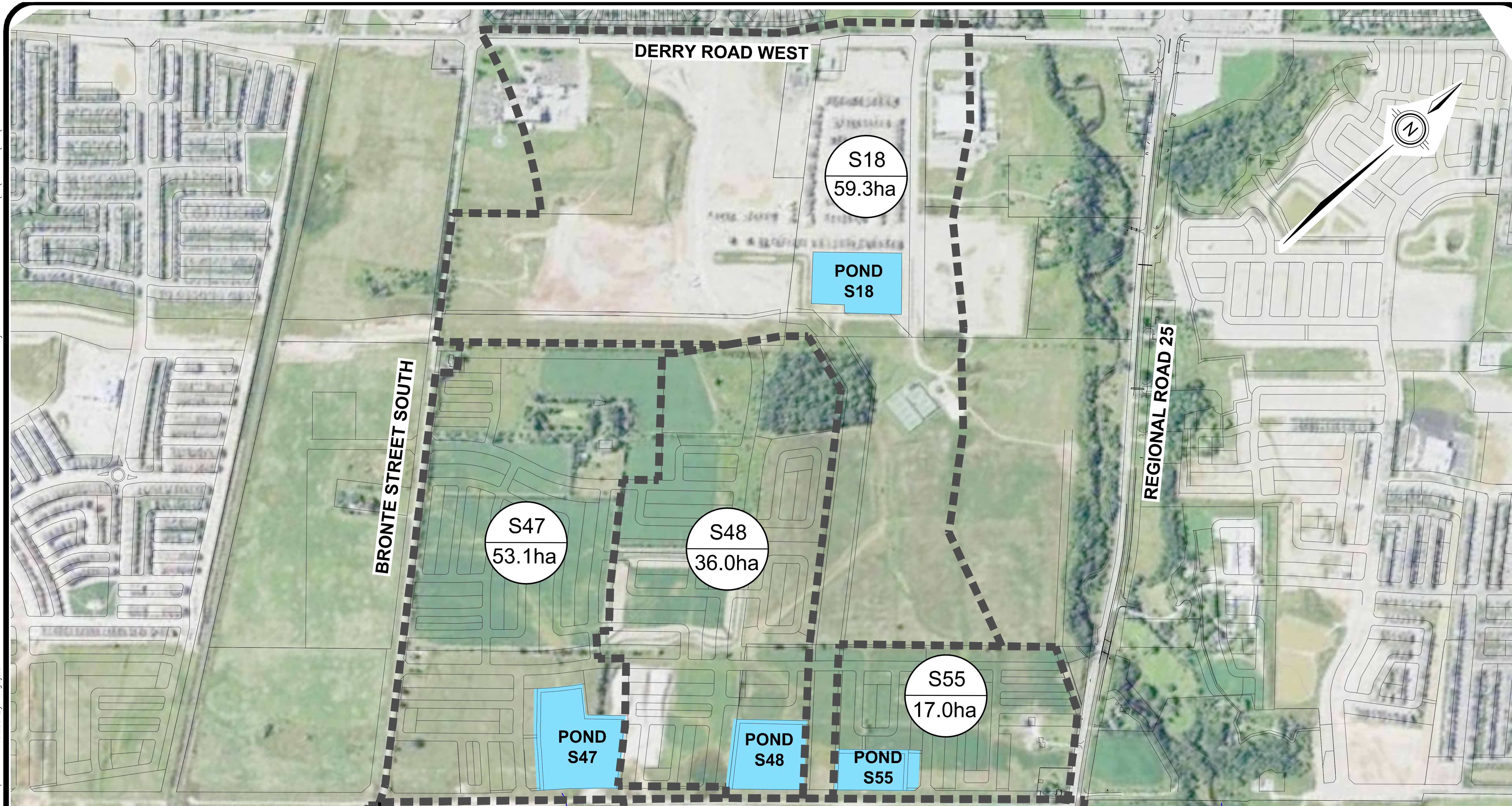
Project Name: SUBWATERSHED IMPACT STUDY - BLOCK 2

Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale	MTE Project No.	Date	Layout No.
1:7500	10477-100	JUNE 2015	FIG 4.2

Project: 10477-100 CAD: P:\P\10477\LD\10477-100-ST1.DWG

June 30, 2015 — 5:43 p.m. — Plotted By: mgojsic



LEGEND

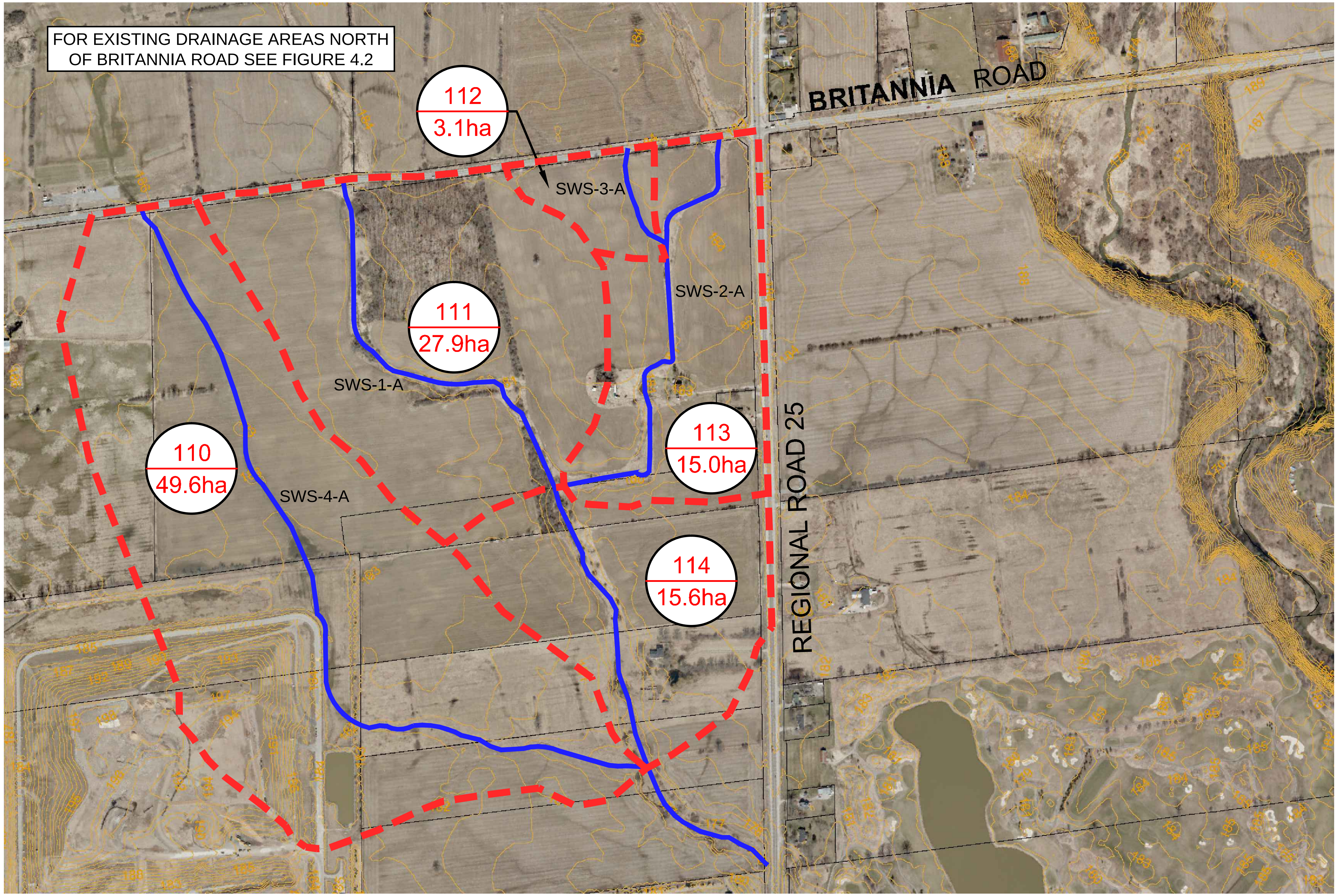
101 — CATCHMENT ID
22.3ha — CATCHMENT AREA IN HECTARES
CATCHMENT LIMIT
EXISTING SWM FACILITY

FIGURE 4.3 - EXTERNAL CONTRIBUTING DRAINAGE AREAS

MTE
Engineers | Scientists | Surveyors

Project Name	
SUBWATERSHED IMPACT STUDY - BLOCK 2	
Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale.	MTE Project No.	Date	Layout No.
1:7500	10477-100	JUNE 2015	FIG 4.3



MAPPING SOURCE: HALTON REGION, 2010

LEGEND

101 / 22.3ha — CATCHMENT ID; CATCHMENT AREA IN HECTARES

182 — EXISTING CONTOURS (APPROXIMATE HALTON REGION 2010)

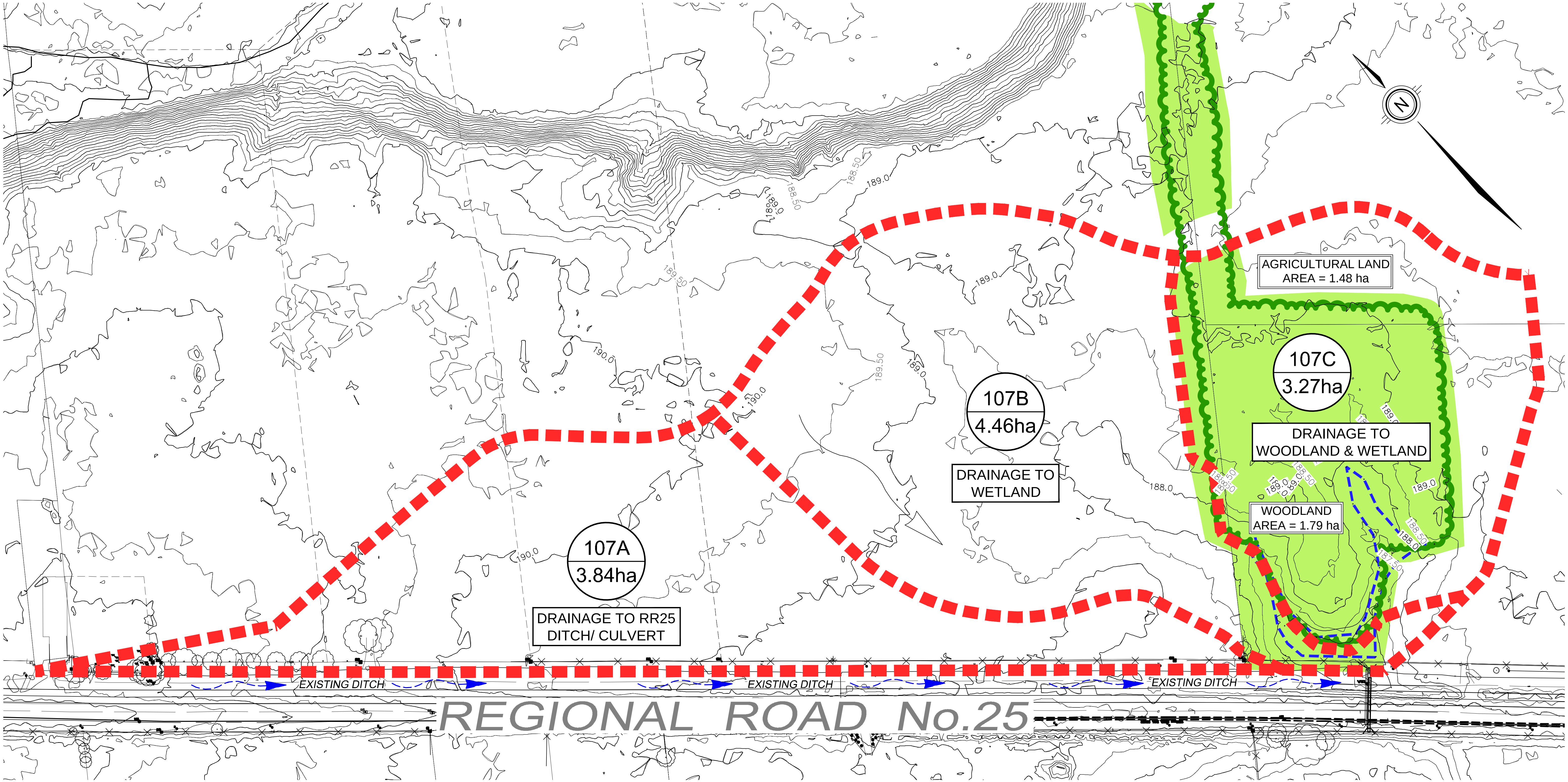
CATCHMENT LIMIT

EXISTING DRAINAGE FEATURES

FIGURE 4.4 - DRAINAGE PATTERNS SOUTH OF BRITANNIA ROAD

MTE
Engineers | Scientists | Surveyors

Project Name			
SUBWATERSHED IMPACT STUDY - BLOCK 2			
Site		Client	
BOYNE SURVEY SECONDARY PLAN		BOYNE SURVEY LAND OWNERS GROUP	
Scale.	MTE Project No.	Date	Layout No.
1:7500	10477-100	JUNE 2015	FIG.4.4



LEGEND

100A / 1.00ha	CATCHMENT ID		EXISTING DITCH
	CATCHMENT AREA IN HECTARES		STAKED DRIPLINE
	CATCHMENT LIMIT		STAKED WETLAND LIMIT
			PROPOSED OPEN SPACE

FIGURE 4.5 - PRE-DE□ELOPMENT WOODLAND □WETLAND DRAINAGE AREAS

MTE
Engineers |Scientists |Surveyors

Project Name			
SUBWATERSHED IMPACT STUDY - BLOCK 2			
Site		Client	
BOYNE SURVEY SECONDARY PLAN		BOYNE SURVEY LAND OWNERS GROUP	
Scale.	MTE Project No.	Date	Layout No.
1:2000	10477-100	AUGUST 2015	FIG.4.5

July 30, 2015 — 4:11 p.m. — Plotted By: mgojsic

P:\P\10477\LD\10477-100-MS15.dwg

Table 4.1 summarizes the existing drainage areas within the Study Area, including drainage towards the Indian Creek Watershed from lands east of Bronte St. S.

TABLE 4.1 – EXISTING CONDITIONS DRAINAGE AREAS

Sub-Catchment	Description	Area (ha)
100	Britannia Road from CN Rail west of Bronte Street South to the Indian Creek watershed limits east of Bronte Street South	3.5
101	Undeveloped drainage to Indian Creek Watershed including Bronte Street South	25.0
Total to Indian Creek Watershed		**28.5**
Ext S47	External Drainage Area to Pond S47 on North side of Louis St. Laurent	53.1
102	Undeveloped drainage area to existing 450 mm diameter culvert associated with SWS-1-A tributary	14.6
103	Undeveloped drainage area directly to SWS-1-A (existing 2.4 m X 1.4 m box culvert)	85.8
Total to SWS-1-A		**153.5**
Ext S18	Miltonbrook subdivision, channel and area to pond S18 on North side of Louis St. Laurent	59.3
Ext S55	External area to pond S55 on North side of Louis St. Laurent	17.0
Ext S48	External area to pond S48 on North side of Louis St. Laurent	36.0
104	Undeveloped drainage area to existing 400 mm diameter CSP culvert associated with SWS-2-A	18.1
105	Undeveloped drainage area directly to SWS-2-A (twin 2.4 m X 1.2 m box culverts)	68.1
106	Regional Road 25 right-of-way drainage to SWS-2-A	5.7
107	Undeveloped drainage area east of Regional Road 25 associated with SWS-2-A tributary	24.2
Total to SWS-2-A		**228.4**
Total		**410.4**

4.2 Existing Stream Conditions

Stream Channel Assessment

A characterization of tributaries SWS-1-A and SWS-2-A was completed by Aqualogic Consulting as input to this report. Appendix A8 contains the report entitled "Natural Channel and Wetland Corridor Preliminary Design, Sixteen Mile Creek Tributaries SWS-1-A and SWS-2-A" (Aqualogic Consulting, March 2014). The report documents that the existing watercourses flow through historically altered pathways that have undergone some re-naturalization over time. The active channel has a narrow width to depth ratio, and is difficult to observe with occasionally braided and enclosed (dense groundcover) sections. These conditions may result in fish passage constraints due to the lack of a well-defined central channel. The tributaries are each well connected to the surrounding wide flat floodplain. For a complete discussion of the existing channel characteristics, refer to Appendix A8.

Existing Floodlines and Riparian Storage

Complete existing conditions HEC-RAS hydraulic modeling was assembled for SWS-1-A and SWS-2-A as part of the SUS completed by AMEC Environment and Infrastructure. Associated existing conditions floodplain mapping was also prepared as part of that study, and the figure illustrating cross-section locations along with the delineated flood line limits is included in Appendix C1 (Drawing No. 7).

One of the objectives of the channel design is to provide an incremental riparian flood storage balance between existing and proposed conditions. The riparian storage for each of the analyzed events (return period events and Regional Storm event) is assumed to be equal to the volume of water under the floodline during proposed conditions peak flow conditions. The storage within a particular channel reach for the existing condition is obtained directly from the storage reported in the HEC-RAS summary output.

The previously completed existing conditions modeling by AMEC, along with flow data inputs refined by AMEC in 2015, was used to determine the existing riparian flood storage within the two creek systems. As per standard practice, the riparian floodplain and riparian storage calculations were done using the HEC-RAS modeling without any man made barriers (culverts/road crossings) in place.

Results of the existing conditions riparian storage analysis are summarized with the proposed conditions results in Section 4.6 of this report.

4.3 Stormwater Management Guidelines

The stormwater management and channel/crossing design criteria and requirements related to the subject development were obtained primarily from the *Final Functional Stormwater and Environmental Management Strategy* (FSEMS; AMEC Foster Wheeler, May 2015), and the *Final Milton Urban Expansion Conceptual Fisheries Compensation Plan Boyne Survey Area – Milton Phase 3* (CFCP; AMEC Environment and Infrastructure, May 2015). Additional information was obtained from the *Engineering and Parks Standards for the Town of Milton* (August 2010).

The following summarizes the general design criteria applicable to the stormwater drainage system within the Study Area:

Minor/Major Drainage System

1. Urban drainage systems (curb/gutter/storm sewers) are preferred by the Town of Milton.
2. The minor drainage system should be designed for the 5-year storm event using the Rational Method and Town of Milton IDF curves.
3. Generally, the obvert of storm sewers is to be placed 1.5 m below the finished grade at the centerline of the road or at a minimum 1.0 m below the dwelling basement floor elevation.
4. The major system should be designed to accommodate runoff exceeding the capacity of the minor system for flows up to the 100-year return frequency. The major system should be contained within road allowances, swales, drainage channels, designated blocks and ponds.

Stormwater management facilities

1. Where possible, SWM facilities should be integrated into or adjacent to open space areas, or natural systems including proposed linkage corridors and watercourses.
2. The drainage area to SWM facilities should be limited to a maximum of 40 to 80 ha.
3. Alternative on-site control stormwater management measures may be considered for developments with a contributing drainage area of less than 5.0 ha.
4. Thermal mitigation practices should be incorporated into all stormwater management facilities (eg. bottom draw, riparian plantings, cooling trenches, deeper outlet pool sumps).
5. The Town of Milton currently prefers that stormwater management facilities be designed as off-line hybrid wet pond/wetland systems in accordance with the "enhanced" protection level for the receiving watercourse as defined by the Ministry of the Environment Stormwater Management Planning and Design Manual, published in March 2003. It is however recognized that through the development process alternative end-of-pipe facilities (i.e. wet ponds and wetlands) may be proposed and implemented based on site-specific criteria.
6. A site-specific rationale for the location and type of SWM facility was established within the 2015 CFCP, 2015 FSEMS and previous studies as follows (as previously noted subsequent design stages may propose alternative facility types):
    a) Where possible, integrate SWM facilities into or adjacent to open space areas, or natural systems including proposed linkage corridors and watercourses.
    b) Adopt a philosophy of hybrid SWMPs for those facilities cited in (a), and wet pond SWMPs for those facilities located in the urban landscape where they are relatively isolated from terrestrial/watercourse habitats or in tableland settings.

c) Generally locate communal SWM facilities at or near changes in land use and outlets at watercourses to be maintained.

7. The required permanent storage should be determined based on Table 3.2 from the March 2003 Stormwater Management Planning and Design Manual, based on the imperviousness level of the contributing drainage area and subtracting 40 m^3/ha required for extended detention storage.
8. The depth of extended detention storage within the wetland portion of the SWM facility should not typically exceed 1 metre since some plants cannot withstand prolonged water level fluctuations greater than 1 metre. Where extended detention depths greater than 1 metre are required, the planting strategy should be designed in accordance with the increased depth requirements.
9. Hybrid Wet Pond/Wetland systems have 50-60% of their permanent pool volume in deeper portions of the facility (e.g., forebay, wet pond).
10. The additional storage volume necessary to mitigate impacts on peak flow rates (flooding) should be based on the flood control characteristics listed in Table 4.2.

TABLE 4.2 - GENERAL DESIGN CRITERIA – FLOOD CONTROL CHARACTERISTICS

Characteristic		SWS-1-A	SWS-2-A
Unitary Flood Control Storage (m^3/Imp. ha)	Extended Detention (Erosion Control)	400	400
	Up to 25-Year Stage	650	600
	Up to 100-Year Stage	950	825
	Regional Storm	1825	1450
Unitary Controlled Flow Rate (m^3/s/ ha)	Extended Detention (Erosion Control)	0.0006	0.0006
	Up to 25-Year Stage	0.010	0.020
	Up to 100-Year Stage	0.035	0.050
	Regional Storm	0.052	0.070

**Data included in Table 4.2 is based on hydrologic verification for Block 2 completed by Amec (refer to June 15/15 correspondence included in Appendix C2). This data includes the impact of the Indian Creek diversion to SWS-1-A.*

11. On-line storage for peak flow control during the Regional Storm event is acceptable in principle if it is demonstrated that impacts to water temperature, terrestrial and aquatic passage, and stream morphology can be adequately managed. Preliminary Regional Storm on-line flood control volume requirements have been estimated within the FSEMS to be 91,000 m^3 and 57,000 m^3 for areas draining to SWS1-A and SWS2-A respectively.
12. A stormwater management facility may be permitted within the Regional Storm floodplain if there is sufficient technical justification and it meets the following requirements:
    a. The facility will not be located within a confined valley;
    b. The facility will be located outside of the 1:100 year floodplain;
    c. The facility will be located outside of the 1:100 year meander belt allowance and a 6 metre erosion access allowance;

d. There will be no loss of floodplain storage or conveyance, due to the removal of fill from the floodplain or through an incremental balanced cut and fill analysis. Flood storage provided by the facility itself is excluded from the floodplain storage; and
e. All other recommended Ministry of Environment guidelines.

13. The design of the SWM facilities should conform to the March 2003 MOE design manual. The permanent pool in the proposed wet ponds should be no greater than 1.5 m in depth. The normal water level depth in the proposed wetlands should be no greater than 0.3 m with the exception of the deep pool outlet area. The active storage volume should have a maximum fluctuation of 1.8 m (100 year event).
14. Maximum 5:1 side (overall) slopes (in areas 3 m above and 3 m below the permanent pool elevation), with minimum 4:1 slopes elsewhere.
15. A 7.5 m buffer is required around those portions of the SWM facility that are adjacent to private land. The buffer is to be graded as per the Town of Milton standard buffer drawing. (See Appendix C1).
16. Facilities to include forebays designed to provide required settling and dispersion performance.
17. Facilities shall provide maintenance/inspection access to the inlet, forebay and outlet locations.

Road Crossings

1. Natural substrate through open footing design or through the use of an embedded culvert invert to a depth of 0.5m preferred (minimum 0.3m).
2. Low flow channel to be provided through each crossing, which may involve staggering the depth of culvert inverts (i.e. multiple culvert crossings to promote low flow through a single culvert).
3. Minimum span recommended to be approximately three times the proposed bankfull width in order to maintain natural channel form.
4. Road crossings will need to accommodate the 100 year erosion rate, as well as satisfying hydraulic criteria for freeboard and depth of overtopping during the Regional storm event, and consider wildlife passage for small mammals, amphibians, and reptiles.

4.4 Stormwater Management Plan Overview

Through the conceptual SWM design process, potential SWM Best Management Practices (BMPs) were reviewed for incorporation into development design. SWM BMPs are specific measures to manage the quality of urban runoff to mitigate drainage impacts. Alternative SWM BMPs were considered and are as follows:

- Infiltration measures such as lot level infiltration galleries, infiltration trenches along the conveyance system, or end-of-pipe infiltration basins;
- Source control measures such as grassed swales, vegetative filter strips, or pervious paving materials; and
- Detention measures such as extended detention SWM facilities.

In determining the recommended SWM plan, each alternative was evaluated on the basis of physical constraints and effectiveness associated with their implementation. The review indicated that:

- The functional purposes of low impact development techniques (LID) are to maximize infiltration and reduce runoff, and have the effect of minimizing the impact on the natural hydrologic regime;
- Based on the geological conditions encountered during the drilling program as discussed in Section 3, on-site soils are considered to be of low permeability and are not ideal for the implementation of infiltration facilities. However, it is recognized that even in these types of soils, a variety of infiltration measures can be implemented to assist in supplementing post development groundwater recharge. “Active” LID measures are defined as constructed facilities such as trenches, basins, or galleries that have a primary function related to stormwater management (e.g. storage, treatment, infiltration, etc.). “Passive” LID measures on the other hand are not specific facilities or infrastructure, rather they are practices applied generically to a development landscape (e.g. increased topsoil thicknesses, roof downspout disconnection).
- There are various “active” infiltration methods that can be used to increase infiltration including construction of filter swales adjacent to parking lot pavement, bio-retention facilities, rain gardens, green roofs, permeable pavement, and subsurface infiltration trenches, basins and galleries. In general the MOECC suggests that for these types of LID (infiltration) measures to be effective, soils should have a minimum infiltration rate of 15 mm/hour. The infiltration rates in this area are less than 5 mm/hour. Therefore, the effectiveness of these types of LID measures is difficult to predict for the Study Area. In addition, these types of LID measures are generally more costly to install, and require some degree of ongoing maintenance.
- There are also a number of “passive” LID measures that can be considered to increase infiltration such as increased topsoil thicknesses, directing roof water to grassed areas and rear yards, and grading designs incorporating long side and rear yard swales. These types of LID measures are relatively inexpensive to install, and do not require any ongoing maintenance. In addition, they can be used over a widespread area resulting in a significant impact
- A number of potential “passive” LID measures have been short-listed within the FSEMS as being potentially suitable for implementation within the Boyne Survey. The short-listed techniques are listed below, and are recommended for implementation within the Study Area:
    - Increased topsoil depth (0.45 m) within appropriate green/open space areas (schools, parks, residential rear yards). Increased topsoil depth used to promote infiltration is to conform to Town of Milton Standards.
    - Roof leader discharge to grade. Runoff from roof leaders can be directed to grassed areas/swales in most areas, where development conditions permit, in order to reduce the total directly connected impervious coverage.

- The potential use of other measures discussed in Section 3 of this report (e.g. filter swales adjacent to parking lot pavement, bioretention facilities, rain gardens, green roofs, and permeable pavement) should be explored for public (i.e. park) and institutional (i.e. school) areas as well as private condominium and commercial properties, where demonstrated to be technically feasible;
- Opportunities to incorporate LID approaches generally require a detailed level of site specific analysis and should be further explored at the functional and detailed design stages;
- Best practices for soil management should be incorporated into final design specifications for the development lands. Specifically, the specifications should address the expected widespread implementation of increased topsoil depth including required levels of organic content and the minimum depth of uncompacted soil at the surface, along with the proposed methodology of placing topsoil and completing any required amendments prior to or following placement. Any site-specific LID measures implemented through the final design process should include similar specifications for soil and compaction requirements and testing. All topsoil specifications are to conform to Town of Milton standards;
- All drainage from roof areas on properties that abut the drainage channels should be directed to grade within the rear yard in order to provide distributed surface runoff inputs along the entire channel length;
- Extended detention hybrid wet pond/wetland SWM facilities are the primary SWM measure that has conceptually been proposed to service the lands within the Study Area. All facilities within this area that are located adjacent to a watercourse have been conceptually designed as hybrid facilities. As previously noted, hybrid facilities are preferred where facilities are located adjacent to watercourses and other environmental features which provide a linkage function. A wet pond design has been advanced for the small SWM facility located east of Regional Road 25, which is physically separated from the drainage corridor. Through the functional and detailed design stages, alternative facility design configurations (e.g. wet pond facilities) can also be explored. As discussed further in a subsequent sub-section, provision has also been made for one dry-pond SWM facility in Catchment 411, to be implemented only in the event that other on-site quantity control measures are deemed infeasible.

The proposed stormwater management plan for the Study Area was developed considering the desire:

- To maintain as closely as possible the existing conditions drainage areas to each of SWS-1-A and SWS-2-A;
- To minimize the long-term stormwater infrastructure burden on the municipality by redirecting runoff from the western portion of Block 2 from Indian Creek to Sixteen Mile Creek tributaries SWS-1-A and SWS-2-A;
- To improve the stormwater management provided to runoff from Bronte Street South and Britannia Road adjacent to Block 2 to the extent feasible, by routing it through SWM facilities within Blocks 1 and 2;

- To limit the maximum drainage area to individual SWM facilities to between 40 – 80 ha and;
- To provide integration of SWM facilities with other open space areas.

The enclosed Drawing 4.1 – Stormwater Management Plan, illustrates key elements of the conceptual storm drainage system for the Study Area. A total of five (5) hybrid wet pond/wetland SWM facilities are proposed, each servicing drainage areas of up to approximately 35 ha. A wet pond SWM facility is proposed to service 5.6 ha of development on the east side of Regional Road 25. A provisional dry pond SWM facility may be implemented to service a 2.2ha area in the event that other on-site controls are deemed infeasible.

The proposed facilities will collect minor and major system flows from the proposed development and provide the required water quality, erosion and quantity control for the contributing drainage areas. The construction of the facilities will include a comprehensive planting scheme to appropriately integrate with the adjacent watercourse features. The facilities will serve to enhance stormwater quality, provide flood control, and manage stream erosion, all in accordance with FSEMS design criteria.

The Regional Storm Control Strategy proposed includes 100% off-line controls. Each SWM facility proposed in Block 2 provides the necessary Regional flood storage volume for the drainage area contributing to that SWM facility, based on unitary Regional storage volumes (i.e. m^3 / Impervious hectare) as set out in the hydrologic verification by AMEC. This is discussed further in Section 4.6.

Preliminary design (by others) for the reconstruction of Bronte Street South and Britannia Road in the area of Block 2 has led to the proposed redirection and division of runoff from those roads. Minor flows from Bronte Street South will be redirected from the Indian Creek wastershed into the Sixteen Mile Creek watershed through Block 2 via SWM Facilities 'G' and 'H' and SWS-1-A. Major flows from Bronte Street South will continue to be directed to Indian Creek, through Boyne Block 1. Minor flows from Britannia Road will be redirected to SWM Facilities 'H' and 'J' in Block 2, while major flows from Britannia Road will be discharged directly to SWS-1-A and SWS-2-A downstream of Block 2.

With reference to Drawing 4.1, Table 4.3 outlines the preliminary post-development drainage conditions within the Study Area. It is noted that design of SWM facilities and other infrastructure within sub-catchment 202 (Indian Creek watershed) is to be addressed within the Block 1 SIS. Similarly, stormwater management treatment measures for major runoff from Bronte Street South are to be addressed within the Block 1 SIS.

TABLE 4.3 – POST-DEVELOPMENT DRAINAGE CONDITIONS

Sub-Catchment	Drainage Area Description and Comments	Receiving SWM Facility or Watercourse	Area (ha)	
			Major	Minor
201	Bronte Street South	Minor storm flows to SWM Facility 'G'; major storm flows to Indian Creek via Block 1	2.6	0
202	Major node area with on-site SWM	Drain via Britannia Road drainage system to Indian	1.1	1.1
203	Bronte Street South	Minor storm flows to SWM Facility 'H'; major storm flows to Indian Creek via Block 1	2.4	0
Total Drainage Area to Indian Creek Watershed			**6.1**	**1.1**
Ext S47	External Drainage Area to Pond S47 on North side of Louis St.	SWS-1-A	53.1	53.1
201	Bronte Street South	Minor storm flows to SWM Facility 'G'; major storm flows to Indian Creek via Block 1	0	2.6
203	Bronte Street South	Minor storm flows to SWM Facility 'H'; major storm flows to Indian Creek via Block 1	0	2.4
301	Residential development area	SWM Facility 'G'	44.5	44.5
302	Residential development area	SWM Facility 'H'	34.3	34.3
303	Residential development area	SWM Facility 'K'	23.4	23.4
304A	Green corridor associated with watercourse	SWS-1-A	3.6	3.6
304B	Green corridor associated with watercourse	SWS-1-A	6.4	6.4
304C	Green corridor associated with watercourse	SWS-1-A	2.7	2.7
305	Louis St. Laurent Avenue	SWM Facility S47 (Sherwood Survey)	0.9	0.9
306	Britannia Road	Minor storm flows to SWM Facility 'H'; major storm flows to SWS-1-A downstream of Britannia Road	0	5.2
Total Drainage Area to SWS-1-A via Block 2			**168.9**	**179.1**
306	Britannia Road	Minor storm flows to SWM Facility 'H'; major storm flows to SWS-1-A downstream of	5.2	0
Total Drainage Area to SWS-1-A downstream of Block 2			5.2	0

Sub-Catchment	Drainage Area Description and Comments	Receiving SWM Facility or Watercourse	Area (ha)	
			Major	Minor
Ext S18	Miltonbrook subdivision, channel and area to pond S18 on North side of Louis St. Laurent	SWS-2-A	59.3	59.3
Ext S55	External area to pond S55 on North side of Louis St. Laurent	SWS-2-A	17.0	17.0
Ext S48	External area to pond S48 on North side of Louis St. Laurent	SWS-2-A	36.0	36.0
401	Residential development area	SWM Facility 'I'	30.8	30.8
402	Regional Road 25 right-of-way drainage	Oil-grit separators and drain to SWS-2-A	3.7	3.7
403	Residential development area	SWM Facility 'J'	35.1	35.1
404A	Green corridor associated with watercourse	SWS-2-A	3.8	3.8
404B	Green corridor associated with watercourse	SWS-2-A	6.9	6.9
404C	Green corridor associated with watercourse	SWS-2-A	2.3	2.3
405A	Green area/rear lot area east of Regional Road 25	SWS-2-A	0.4	0.4
405B	Green area/rear lot area east of Regional Road 25	Wetland	2.8	2.8
405C	Roof area east of Regional Road 25, to be directed to Woodland/Wetland	Wetland	0	0.6
406	Residential development area	SWM Facility 'L'	7.0	7.0
407	Major node area with on-site SWM	Drain via Britannia Road drainage system to SWS-2-A	2.4	2.4
408	Regional Road 25 right-of-way drainage	Oil-grit separators and drain to SWS-2-A	2.4	2.4
409	Louis St. Laurent Avenue	SWM Facilities S48 and S55 (Sherwood Survey)	1.5	1.5
410	Britannia Road	Minor storm flows to SWM Facility 'J'; major storm flows to SWS-2-A downstream of	0	1.6
411	Residential Area with on-site SWM	SWS-2-A	2.4	2.4
412	Residential development area	SWM Facility 'I'	2.2	2.2
Total Drainage Area to SWS-2-A via Block 2			**216.0**	**218.2**
410	Britannia Road	Minor storm flows to SWM Facility 'J'; major storm flows to SWS-2-A downstream of	1.6	0
413	Britannia Road	Major and minor storm flows to SWS-2-A downstream of	1.5	1.5

Sub-Catchment	Drainage Area Description and Comments	Receiving SWM Facility or Watercourse	Area (ha)	
			Major	Minor
Total Drainage Area to SWS-2-A downstream of Block 2			**3.1**	**1.5**
501	East of Regional Road 25	Sixteen Mile Creek via Block 3	11.0	11.0
405C	Roof area east of Regional Road 25, to be directed to Woodland/Wetland	Sixteen Mile Creek via Block 3	0.6	0
Total Drainage Area to Sixteen Mile Creek via Block 3			11.6	11.0
Total Study Drainage Area			**410.9**	**410.9**

*** Note:** Minor discrepancy as compared to Existing Drainage Area total exists due to rounding.

Table 4.4 summarizes the estimated directly and indirectly connected impervious areas within each of the catchments that are internal to Block 2 and illustrated on Drawing 4.1. The estimated total impervious fraction listed in Table 4.4 for each catchment has been selected as an initial conservative estimate of what the lumped fraction for each catchment would be. This value should be refined at the detailed design stage, and it is expected that at that time, the calculated impervious fraction for the lands upstream of each individual facility will fall between 55% and 60%.

TABLE 4.4 – POST-DEVELOPMENT IMPERVIOUS FRACTIONS

Sub-Catchment	Estimated Directly Connected Impervious Fraction (%)	Estimated Indirectly Connected Impervious Fraction (%)	Estimated Total Impervious Fraction (%)
301	35	25	60
302	35	25	60
303	35	25	60
304A	0	15	15
304B	0	15	15
304C	0	15	15
305	35	25	60
306	35	35	70
401	35	25	60
402	35	25	60
403	35	25	60
404A	0	15	15
404B	0	15	15
404C	0	15	15
405A	0	10	10
405B	0	10	10
405C	100	0	100
406	35	25	60
407	65	12.5	77.5
408	35	25	60
409	35	25	60
410	35	35	70
411	25	45	70
412	35	25	60

Bronte Street South and Britannia Road

Reconstruction of both Bronte Street South and Britannia Road is expected in the short- to medium-term. Through discussion with staff from the Town and Region, and the Block 1 and Block 2 landowners and their consultants, it has been agreed that runoff from these boundary roads will be treated as follows:

- Minor storm drainage from Bronte Street South will be collected and conveyed to SWM facilities ‘G’ and ‘H’ in Block 2 for treatment;
- Major storm drainage from Bronte Street South will be conveyed to SWM facilities in Block 1 for treatment;
- Minor storm drainage from Britannia Road will be collected and conveyed to SWM facilities ‘H’ and ‘J’ in Block 2 for treatment; and
- Major storm drainage from Britannia Road will be collected and discharged directly to SWS-1-A and SWS-2-A downstream of Block 2.

Any future works for both Bronte Street South and Britannia Road will need to be discussed with Conservation Halton.

Catchment areas associated with Bronte Street South and Britannia Road are shown on Drawing 4.1

Indian Creek Drainage Divide
As summarized in Table 4.1 (Section 4.1), for existing conditions, 22.3ha of land east of Bronte Street South currently drains to the Indian Creek watershed. Under the preliminary proposed development concept, runoff from the majority of these lands will be diverted into the Sixteen Mile Creek subwatershed via the proposed SWM facilities discharging to SWS-1-A.

The concept of diverting runoff from the Indian Creek subwatershed to the Sixteen Mile Creek SWS-1-A subwatershed was assessed by AMEC, and the following conclusions were drawn (refer to October 21/14 correspondence included in Appendix C2):

- Requisite erosion control and flood frequency control for the Block 2 SIS Area can be achieved with the proposed diversion of runoff from the diverted area;
- Diversion of the runoff from the Indian Creek subwatershed would reduce the monthly surface runoff volumes to Indian Creek in the winter, early spring and late fall months;
- Average surface runoff volumes to Indian Creek subwatershed would be above existing levels during the summer months, even with the diversion of runoff to the Sixteen Mile Creek subwatershed; and
- Recognizing that the surface runoff volume to Indian Creek would be above existing levels during summer conditions which are considered the most critical for sustaining downstream aquatic habitat, the above results are considered acceptable.

Additionally, minor flows from portions of Bronte Street South and Britannia Road will be collected and conveyed to the SWS-1-A SWM facilities. Major flows from Bronte Street South will be conveyed to Boyne Block 1 where stormwater management treatment will be provided. Major flows from Britannia Road will be discharged directly to SWS-1-A and SWS-2-A south of Block 2.

SWS-1-A Drainage

The preliminary major and minor drainage areas associated with SWS-1-A upstream of Britannia Road total 168.9 and 179.1 ha in the post-development condition, respectively. This represents a change versus the total existing conditions drainage area due to the redirection of flows from the Indian Creek subwatershed. Development areas draining to SWS-1-A will be treated with the use of two mid-block SWM facilities (Facilities 'G' and 'K') along with one facility along Britannia Road (Facility 'H"). Additionally, minor drainage from portions of Bronte Street South and Britannia Road will be conveyed to SWM Facilities 'G' and 'H' for treatment.

SWS-2-A Drainage

Table 4.3 summarizes the post-development drainage areas directed to SWS-2-A upstream of Britannia Road. Three SWM facilities will be incorporated within this portion of the Study Area to treat the majority of the development area drainage. SWM Facility 'I', located approximately midway between Britannia Road and Louis St. Laurent Avenue, will accommodate drainage from the residential area located northeast of SWS-2-A, along with the development area immediately south of the facility (see Drawing 4.1). SWM Facility 'J', located adjacent to Britannia Road, will be designed to treat drainage from the residential area located west of SWS-2-A and south of the drainage area divide for SWM Facility 'K', as well as minor discharge from a portion of Britannia Road. SWM Facility 'L' will be a wet pond and will address SWM objectives for the residential development area located east of Regional Road 25.

Controlled flows from SWM Facility 'L' will outlet across Regional Road 25 via either the existing 1.8m x 0.9m box culvert or a new storm sewer to be constructed under Regional Road 25. Discharging to a new storm sewer will provide a grading advantage for the lands draining to SWM Facility 'L', in that the new outlet could be constructed lower than the culvert, which will minimize fill requirements. Flows discharging from the SWM facility 'L' emergency spillway would still be conveyed through the culvert and proposed downstream linkage feature to SWS-2-A. The proposed configuration for this storm sewer (illustrated on Figure 4.16 in Section 4.5), has been used in the conceptual SWM design/grading scheme within this report. The storm sewer crossing location is limited by the vertical profiles of the existing large diameter sanitary sewer and watermain on Regional Road 25. While it is noted that discharging to the existing culvert is considered preferable to the Town as it would minimize the potential for infrastructure conflicts within the linkage corridor and would also avoid disturbance to the recently constructed Regional Road 25, the selection of the outlet configuration for this facility (i.e. discharge to a new storm sewer or to the existing box culvert) will need to be completed during subsequent design stages. In order to evaluate the potential new storm sewer crossing, functional design alternatives will need to be prepared in order to confirm a preferred route. The functional design will need to consider the location of the existing infrastructure within Regional Road 25 right of way along with potential conflicts with future local infrastructure in the development area between Regional Road 25 and SWS-2-A, including the storm pipe conveying flows across the linkage. Regional approval would be required prior to implementation of a new storm sewer crossing. To ensure that major flows from the 2.2ha area of Catchment 412 lying south of SWM

Facility ‘I’ do not bypass the SWM facility, the storm sewer proposed to convey flows from across the linkages to SWM Facility ‘I’ will be designed to convey the 100-year flow. Preliminary sizing calculations, included in Appendix C3, indicate that a 750mm dia. pipe at 0.4% slope will be sufficient for this purpose.

Drainage area 407, located northeast of the intersection of Britannia Road and Regional Road 25, will drain to SWS-2-A via an extension of the existing 1200 mm diameter culvert that crosses Regional Road 25 approximately 75 m north of Britannia Road (refer to Figure 4.2). This area consists of approximately 2.4 ha of major node development. Water quality, quantity and erosion controls for the development area are proposed to be provided through the implementation of on-site stormwater management techniques including a combination of some or all of the following:

- Rooftop controls - water quantity;
- Parking lot storage – water quantity;
- Underground storage – water quantity, erosion;
- Dry pond – water quantity, erosion;
- Oil/Grit Separator(s) – water quality;
- Bioswales and vegetated filter strips – water quality, erosion; and
- Bioretention areas – water quality, erosion.

The potential use of these SWM best management practices to enhance water quality and erosion control can be pursued through the functional and detailed design stages for this area.

As is demonstrated by the preliminary assessment of the existing 1200mm diameter culvert crossing Regional Road 25 (i.e. Culvert No. 17) between Drainage Areas 407 and 411 included in Appendix C3, the existing culvert has sufficient capacity to convey the uncontrolled runoff (and, therefore, the controlled runoff) from Drainage Area 407 and Drainage Area 408 (i.e. the Regional Road 25 right of way) across Regional Road 25 without surcharge, under the 100 year and Regional storm events. A summary of the model included in Appendix C3 is included in Table 4.5

TABLE 4.5 – REGIONAL ROAD 25 CULVERT CAPACITY (CULVERT NO. 17)

	Uncontrolled Runoff Flow Rate (m^3/s)	
	100 Year Event	Regional Event
Drainage Area 407	1.713	0.569
Drainage Area 408	1.122	0.372
Combined Drainage Area	2.798	0.927
Culvert No. 17 Capacity	3.800	3.800

Further discussion with, and concurrence from, Halton Region of the above will be required at the detailed design stage, once specific flows entering the Regional Road 25 right of way from the upstream development are known.

The same approach is proposed for drainage area 411, which will outlet directly to SWS-2-A at Britannia Road.

The proposed multiple residential and commercial land uses within Area 407 and 411 are well suited to these types of on-site private measures listed above. The option of providing a small SWM facility within Area 411 was explored, however it was concluded that, because some on-site controls would be required in any event for area 407 to prevent uncontrolled flows from either entering the Britannia Road drainage system or crossing Regional Rd. 25, multiple privately owned SWM systems would provide the most efficient drainage solution.

In the event that the type of development to occur in Drainage Area 411 precludes private, on-site SWM treatment, it will be possible to create a block for a stormwater management facility (i.e. dry pond) at the south end of Drainage Area 411. Preliminary sizing of such a SWM block would be approximately 0.4ha, which could provide quantity and erosion control treatment for runoff from Drainage Area 411 and, if necessary, the area of Drainage Area 412 situated south of SWM Facility 'I'. Connecting Drainage Area 412 to this potential SWM facility would remove the need for a piped major flow crossing beneath the channel linkage immediately south of SWM Facility 'I'. Preliminary design details related to this provisional SWM facility are included in Table 4.6 as 'Provisional SWM Facility 'M'. In the event that a dry pond SWM facility is proposed for drainage area 411, water quality treatment would be provided by an Oil & Grit Separator sized appropriately for the development.

Drainage Areas 402 and 408 (Regional Road 25) will continue to drain directly to channel SWS-2-A. This portion of Regional Road 25 was recently reconstructed and runoff is treated with a number of oil-grit separators.

The total major and minor post-development drainage areas to SWS-2-A upstream of Britannia Road are 216 and 218.3 ha, respectively, which represent a slight reduction from existing conditions (12.4 / 10.1 ha or approximately 5%). The majority of this area will be directed to the Main Branch of Sixteen Mile Creek, with stormwater management treatment to be provided by facilities located to the east of the Study Area.

Woodland/Wetland Area Drainage
As previously outlined within Section 4.1 and illustrated on Figure 4.5, the woodland and wetland features east of Regional Road 25 currently receive drainage from small adjacent areas. The extents of Catchment 107C that are located outside of the woodland will be somewhat reduced in post-development conditions. However, it is recommended that flows from the directly adjacent roof and rear yard areas be directed as overland flow towards the woodland, such that the average annual volume of surface water contributions from the reduced area will be similar to current conditions.

Existing conditions surface runoff contributions to the wetland from the agricultural area located north of the woodland (Catchment 107B) should also be mimicked in the post-development condition, unless further detailed study identifies that this drainage is not required to maintain the form and function of the wetland. It is recommended that unless otherwise justified, the drainage plan for the development located north of the woodland incorporate mitigation measures that will direct the necessary volume of runoff to the feature such that the average annual volume of surface water contributions to the wetland will be similar to the current contributions from Catchment 107B. A runoff balance to the wetland will require the direction of additional volume (beyond adjacent roof and rear yard areas). Collecting roof runoff from an estimated roof area of approximately 0.62 ha would supplement the surface water deficit (representing approximately 504m of frontage), as shown on Figure 4.6. Supporting calculations are included in Appendix C4. This demonstrates a viable alternative for supplementing the surface water deficit to the wetland. Verification of this water balance will be required at the detailed design stage. Other stormwater conveyance alternatives to a roof water collection system could also be explored at that time, and would be subject to approval by the Town and Conservation Halton. Verification of this water balance will be required at the detailed design stage.

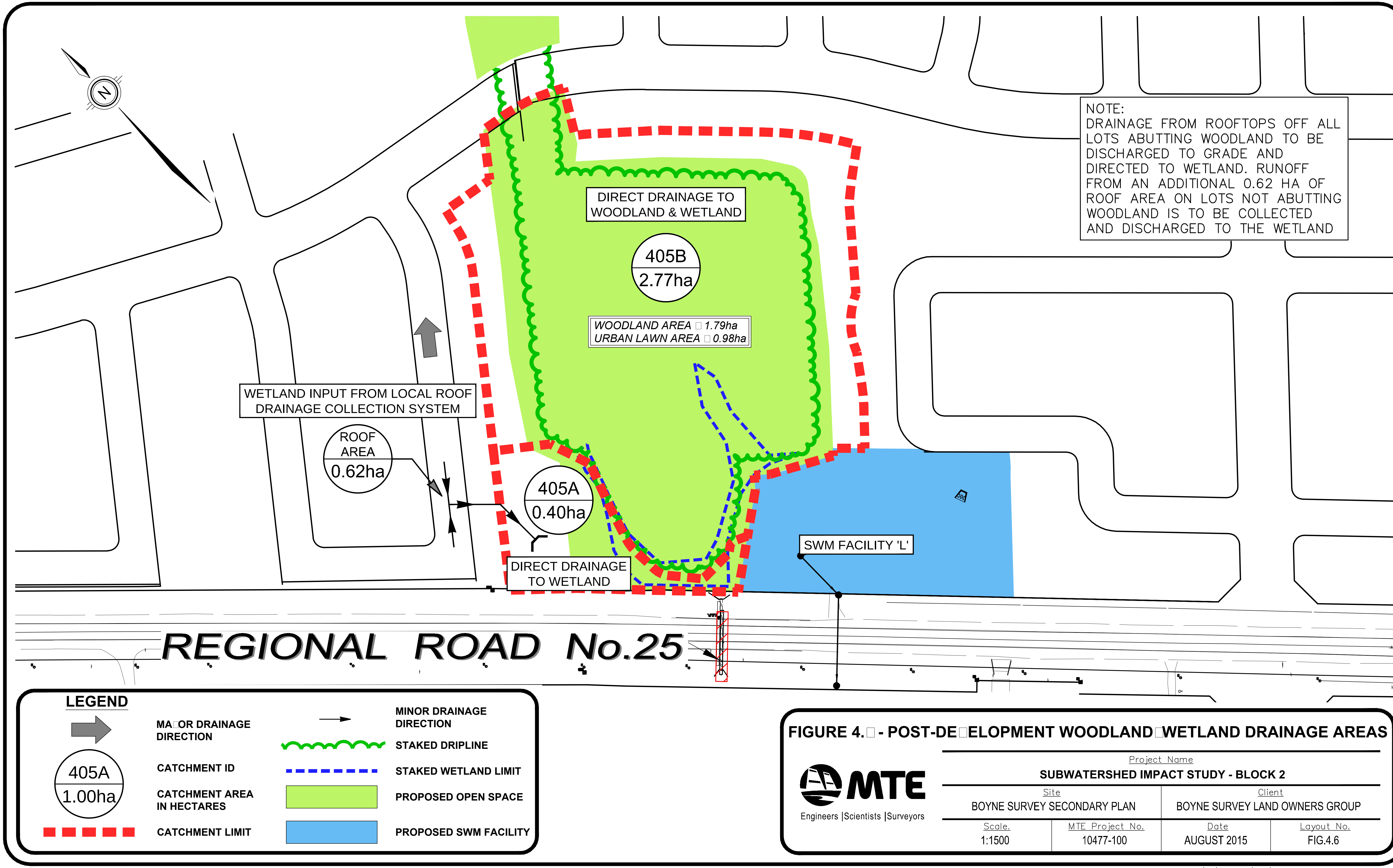
An additional monitoring well is proposed to be installed adjacent to the wetland, prior to draft plan applications on the east side of Regional Road 25, to confirm if there are any groundwater contributions to the wetland.

4.5 Conceptual Stormwater Management Facility Design

As previously identified, stormwater quality and quantity control within the development is proposed to be achieved by the implementation of hybrid wet pond/wetland facilities (SWM Facilities ‘G’, ‘H’, ‘I’, ‘J’ and ‘K’) and a wet pond facility (SWM Facility ‘L’), which will provide permanent and extended detention storage for stormwater quality and erosion control, and additional storage for flood control up to the Regional storm, as well as a provisional dry pond SWM facility (provisional SWM Facility ‘M’) that will provide quantity controls. SWM facility ‘L’ has been conceptually designed as a wet pond facility due to the relatively small size of the contributing drainage area, which is less than one fourth of the area contributing drainage to any of the proposed hybrid SWM facilities. Since SWM facilities for small drainage areas are very inefficient in terms of land use as compared to those for larger drainage areas, a wet pond SWM facility is proposed for SWM Facility ‘L’ in order that it can be as efficient as possible in terms of block size/drainage area ratio as compared to the other facilities (e.g. 11% for SWM Facility ‘I’ vs 6-7% for other facilities). Water quality treatment provided by wet pond SWM Facility ‘L’ will be designed to meet the same MOE “Enhanced” criteria that would be required of a hybrid wet pond/wetland facility. Provisional SWM Facility ‘M’ can be implemented in the event that other on-site water quantity controls within Drainage Area 411 are deemed infeasible. In any event, water quality controls in Drainage Area 411 are to be provided by an oil/grit separator or other on site control.

Each hybrid facility will generally consist of a sediment forebay, a wet pond and a wetland permanent pool area. SWM facility ‘L’ will contain a sediment forebay and wet pond only. Provisional SWM facility ‘M’ would include only a dry cell quantity control basin. Each facility will also include an active storage component for erosion and flood flow control.

The preliminary design of the proposed facilities is based on the criteria and guidelines previously outlined in Section 4.3. The water quality control volumes have been calculated in accordance with the “Enhanced” protection level for the receiving watercourse as defined in the 2003 MOE Stormwater Management Practices Planning



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and Design Manual. The preliminary design parameters and storage requirements for the proposed stormwater management facilities are presented in Table 4.6.

TABLE 4.6 – PRELIMINARY SWM FACILITY DESIGN PARAMETERS

SWM FACILITY ID	SWM FACILITY 'G'	SWM FACILITY 'H'	SWM FACILITY 'I'	SWM FACILITY 'J'	SWM FACILITY 'K'	SWM FACILITY 'L'	PROVISIONAL SWM FACILITY 'M'
SWM Block Area (ha)	2.83	2.25	2.08	2.02	1.70	0.52	0.42
Drainage Area (ha) (Major/Minor)	44.5 / 47.1	34.3 / 41.9	33.0 / 33.0	35.1 / 36.7	23.4 / 23.4	7.0 / 7.0	2.4 / 2.4
Impervious Fraction (%)	60	60	60	60	60	60	70
Required Permanent Storage (m^3)	5,558	4,944	3,894	4,331	2,761	1,134	0
Provided Permanent Storage (m^3)	5,611	12,778	3,894	4,459	2,885	1,424	0
Conceptual Permanent Water Level (m)	184.30	183.10	183.80	182.50	185.10	185.00	182.65
Required Extended Detention Release Rate (m^3/s)	0.0283	0.0251	0.0198	0.0220	0.0140	0.0042	0.0014
Extended Detention Volume Required (m^3)	11,304	10,056	7,920	8,808	5,616	1,680	672
Conceptual Extended Detention Level (m)	185.40	183.86	184.58	183.30	185.78	185.88	183.37
Required 25 Year Release Rate (m^3/s)	0.4450	0.3430	0.6600	0.7020	0.2340	0.1400	0.0480
25 Year Flood Control Required (m^3)	17,355	13,377	11,880	12,636	9,126	2,520	1,008
Conceptual 25 Year Storage Level (m)	185.91	184.08	184.93	183.61	186.15	186.21	183.51
Required 100 Year Release Rate (m^3/s)	1.5575	1.2005	1.6500	1.7550	0.8190	0.3500	0.1200
100 Year Flood Control Required (m^3)	25,365	19,551	16,335	17,375	13,338	3,465	1,386
Conceptual 100 Year Storage Level (m)	186.50	184.47	185.29	183.96	186.56	186.54	183.67
Required Regional Storm Release Rate	2.3140	1.7836	2.3100	2.4570	1.2168	0.4900	0.1680
Regional Flood Control Required (m^3)	48,728	37,559	28,710	30,537	25,623	6,090	2,436
Conceptual Regional Storage Level (m)	187.80	185.50	186.18	184.83	187.56	187.28	184.70

In addition to the above noted parameters, the design of the facilities is to consider the following:

- Each facility will include an emergency outlet for the conveyance of flows for storms larger than the Regional design event.
- 7.5 m buffers are to be incorporated within the SWM facility designs adjacent to proposed private property.
- A 4-metre wide maintenance access route is to be provided from a municipal road with a maximum slope of 10:1 and a maximum crossfall of 2%. It will be used to facilitate the access to the forebay and outlet structure for maintenance. As required by Conservation Halton, the maintenance access at SWM Facility 'I' will include topsoil and vegetated surface treatment in order to provide a naturalized surface to support the linkage function.
- The MOE Guidelines recommend that a sediment drying area should only be incorporated into SWM facility design when it would impose no additional land requirement. As such, sediment drying areas have not been included in the preliminary design of the SWM facilities within the study area. Conversely, when sediment removal is required, it is recommended that the forebay be drained and the sediment be vacuum excavated for transport to a suitable disposal facility.
- Incorporation of thermal impact mitigation measures as best management practices.These measures are intended to provide the conditions within the watercourses to support healthy warm water fish communities. The following thermal impact mitigation measures are proposed for consideration for the SWM facilities within Block 2:
    - Increasing the pool depth to approximately 3.0m below the permanent pool elevation in the vicinity of the outlet pipe. This deep pool should be sized to provide a reservoir of cool water, which will be discharged from the pond during the first 5mm of an event (MNRF has found this approach has been successful in reducing water temperatures)
    - Outlet structures incorporating bottom draws/reverse sloped pipes
    - Increasing canopy cover within the SWM facility (particularly along the west and south sides)
    - Cooling trenches between pond outlet and watercourses - It is preferred that if cooling trenches are required that they be located within the SWM blocks. If necessary, but not preferred, cooling trenches could also be located adjacent and parallel to storm sewer outlets within the channel corridors (including the channel linkage between RR25 and SWS-2-A) subject to a design approved by The Town of Milton.
    - Enhancement of riparian vegetation along the drainage path between the SWM facility outlet and the receiving watercourse.
    - LID measures that promote infiltration to groundwater should be encouraged as these will help to reduce run off to the SWM facilities, and reduce the volume of stormwater passing through the SWM facilities. In addition, it can help to maintain groundwater contributions to the watercourses where applicable.

Selection of the appropriate mitigation measures for a specific pond and location should be completed at the detailed design stage.

- Outlet control device designs are to consider flood flow elevations within the channel (i.e. tailwater conditions).

The location of the SWM facilities are shown on Drawing 4.1 – Stormwater Management Plan and the geometry and conceptual design details for each of the proposed SWM facilities are shown on Figures 4.7 through 4.13. As illustrated on Figures 4.7 through 4.11, internal berming is recommended within the main pool of SWM facilities ‘G’, ‘H’, ‘I’, ‘J’ and ‘K’ in order to provide a minimum average length to width ratio of 5:1.

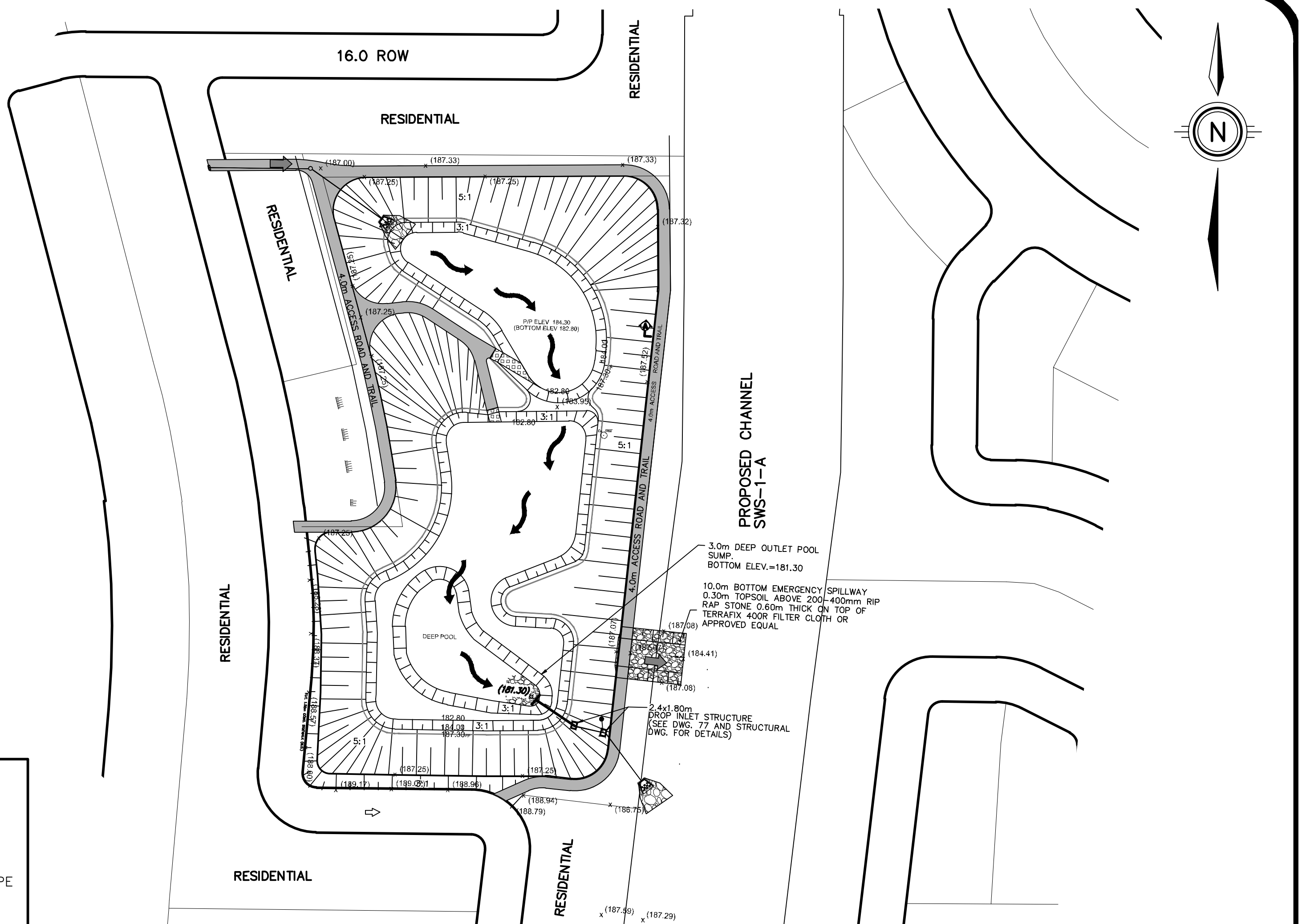
To achieve optimal quantity control performance of SWM facilities under the Regional storm event without overcontrolling under the minor storm events, the conceptual design of the outlet structures includes utilization of flow control orifices in series. An example of this design, for SWM Facility ‘K’, is illustrated in Figures 4.14 and 4.15. Outlet flow design calculations are included in Appendix C5.

Conceptual design for conveyance of discharge from SWM Facility ‘L’ across Regional Road 25, as discussed in Section 4.4, is shown on Figure 4.16.

NOTES:

1. DETAILED DESIGN TO INCLUDE RATIONALE AS TO HOW THERMAL MITIGATION OPTIONS DESCRIBED IN SECTIONS 2.8.6 AND 4.5 WERE CHOSEN OR NOT CHOSEN.
2.
3. PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.

FACILITY CHARACTERISTICS	
PERMANENT WATER LEVEL	184.30m
PERMANENT POOL VOLUME	5,611m³
EXTENDED DETENTION LEVEL	185.40m
EXTENDED DETENTION VOLUME	11,304m³
100 YEAR WATER LEVEL	186.50m
100 YEAR WATER VOLUME	25,365m³
REGIONAL WATER LEVEL	187.80m
REGIONAL WATER VOLUME	48,728m³
BLOCK SIZE	2.83ha
LENGTH TO WIDTH RATIO	6.8 : 1



LEGEND

x(189.79)	PROPOSED SPOT ELEVATIONS
(TOP) 3:1 (BOTTOM)	EMBANKMENT (SLOPE AS NOTED)
	PROPOSED STORM SEWER
	FLOW DIRECTION
	MANHOLE
	HEADWALL(S)
	RIP RAP
	OVERLAND FLOW ROUTE

FIGURE 4.7 - CONCEPTUAL SWM POND "G"

MTE
Engineers | Scientists | Surveyors

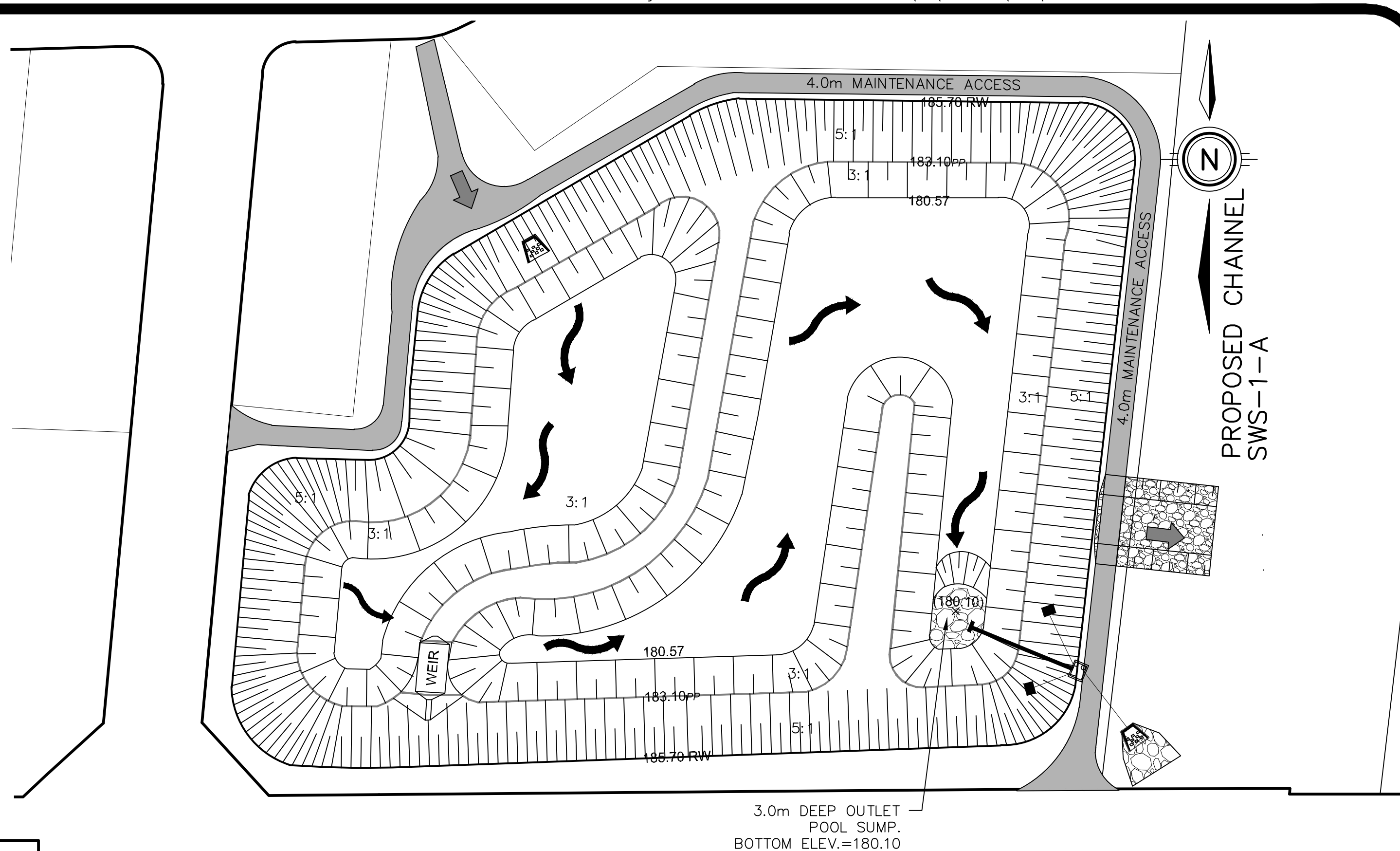
Project Name
SUBWATERSHED IMPACT STUDY - BLOCK 2

Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale.	MTE Project No.	Date	Layout No.
1:2500	10477-100	AUGUST 2015	M4-7-POND-G

FACILITY CHARACTERISTICS

PERMANENT WATER LEVEL	183.10m
PERMANENT POOL VOLUME	12,778m³
EXTENDED DETENTION LEVEL	183.86m
EXTENDED DETENTION VOLUME	10,056m³
100 YEAR WATER LEVEL	184.47m
100 YEAR WATER VOLUME	19,551m³
REGIONAL WATER LEVEL	185.50m
REGIONAL WATER VOLUME	37,559m³
BLOCK SIZE	2.25ha
LENGTH TO WIDTH RATIO	9.2 : 1



NOTES:

1. DETAILED DESIGN TO INCLUDE RATIONALE AS TO HOW THERMAL MITIGATION OPTIONS DESCRIBED IN SECTIONS 2.8.6 AND 4.5 WERE CHOSEN OR NOT CHOSEN.
2. PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.

LEGEND

x(189.79)	PROPOSED SPOT ELEVATIONS
(TOP) 3:1 (BOTTOM)	EMBANKMENT (SLOPE AS NOTED)
	PROPOSED STORM SEWER
	FLOW DIRECTION
	MANHOLE
	HEADWALL(S)
	RIP RAP
	OVERLAND FLOW ROUTE

FIGURE 4.8 - CONCEPTUAL SWM POND H

MTE
Engineers | Scientists | Surveyors

Project Name			
SUBWATERSHED IMPACT STUDY - BLOCK 2			
Site: BOYNE SURVEY SECONDARY PLAN		Client: BOYNE SURVEY LAND OWNERS GROUP	
Scale.	MTE Project No.	Date	Layout No.
1:1500	10477-100	AUGUST 2015	AA M4-8- POND-H

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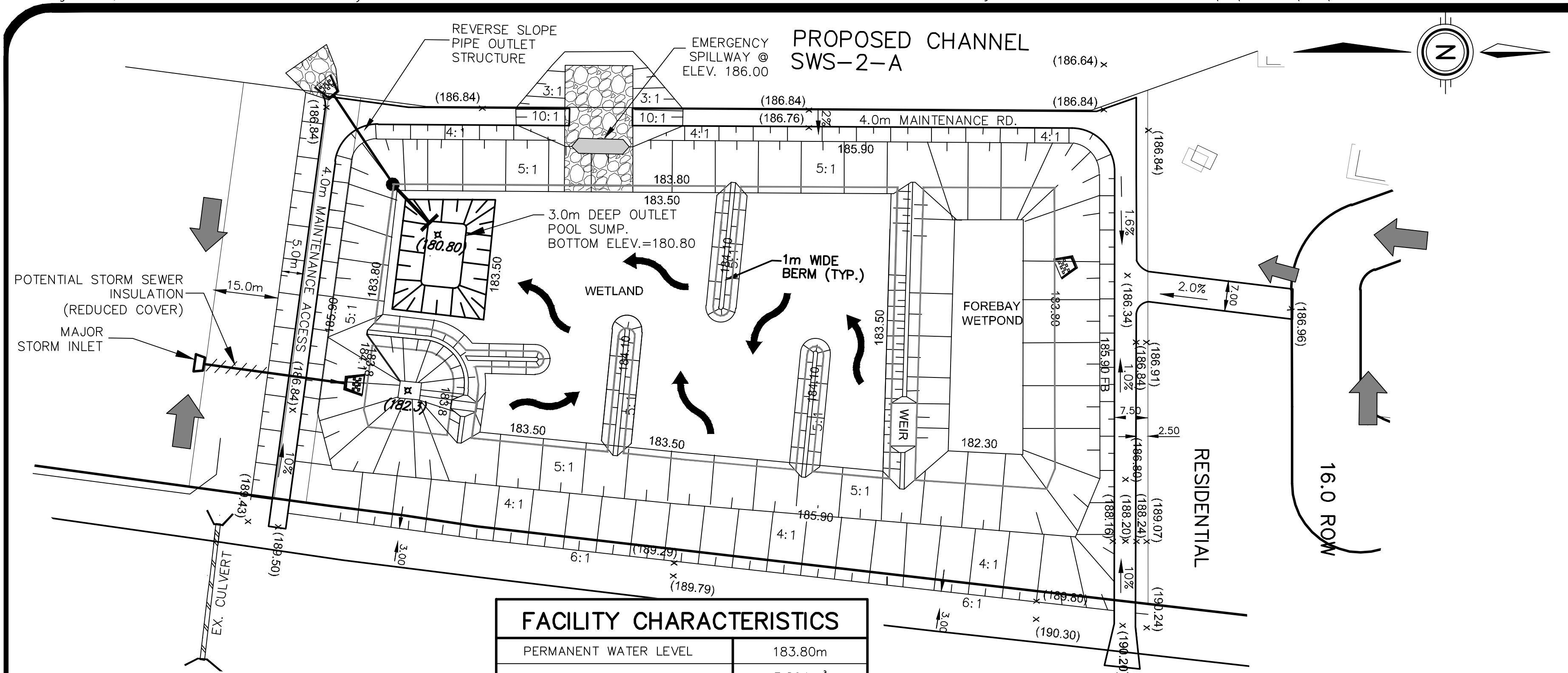


FACILITY CHARACTERISTICS

PERMANENT WATER LEVEL	183.80m
PERMANENT POOL VOL	3,894m³
EXTENDED DETENTION LEVEL	184.58m
EXTENDED DETENTION VOL	7,920m³
100 YR WATER LEVEL	185.29m
100 YEAR WATER VOLUME	16,335m³
BLOCK SIZE	2.08ha
LENGTH TO WIDTH RATIO	9.2 : 1
REGIONAL DETENTION LEVEL	186.18m
REGIONAL DETENTION VOL	28,710m³

NOTES:

1. DETAILED DESIGN TO INCLUDE RATIONALE AS TO HOW THERMAL MITIGATION OPTIONS DESCRIBED IN SECTIONS 2.8.6 AND 4.5 WERE CHOSEN OR NOT CHOSEN.
2. PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.

LEGEND

- ×(189.79) PROPOSED SPOT ELEVATIONS
- (TOP) 3:1 (BOTTOM) EMBANKMENT (SLOPE AS NOTED)
- PROPOSED STORM SEWER
- PROPOSED INSULATION OVER STORM SEWER
- FLOW DIRECTION
- MANHOLE
- HEADWALL(S)
- RIP RAP
- OVERLAND FLOW ROUTE

FIGURE 4.9 - CONCEPTUAL SWM POND I

MTE Engineers | Scientists | Surveyors

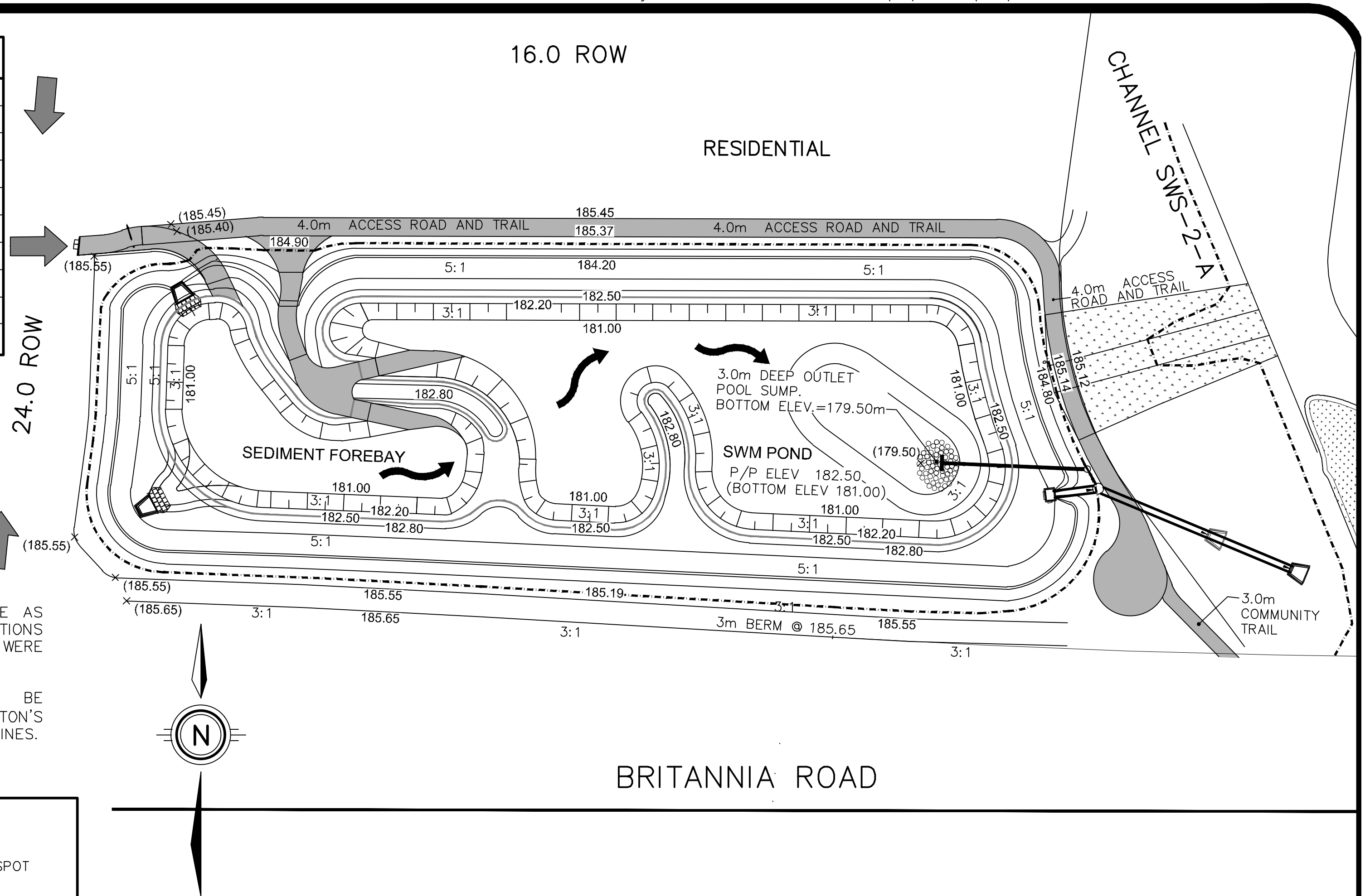
Project Name			
SUBWATERSHED IMPACT STUDY - BLOCK 2			
Site: BOYNE SURVEY SECONDARY PLAN		Client: BOYNE SURVEY LAND OWNERS GROUP	
Scale.	MTE Project No.	Date	Layout No.
1:1500	10477-100	AUGUST 2015	M4-9 POND-I

August 25, 2015 — 9:57 a.m. — Plotted By: rmoir

Project: 10477-100 CAD: P:\P\10477\LD\10477-100-MS5-SWM PONDS.DWG

FACILITY CHARACTERISTICS

PERMANENT WATER LEVEL	182.50m
PERMANENT POOL VOL	4,431m³
EXTENDED DETENTION LEVEL	183.30m
EXTENDED DETENTION VOL	8,808m³
100 YR WATER LEVEL	183.96m
100 YEAR WATER VOLUME	17,375m³
REGIONAL WATER LEVEL	184.83m
REGIONAL DETENTION VOLUME	30,537m³
BLOCK SIZE	2.03ha
LENGTH TO WIDTH RATIO	7.4 : 1



NOTES:

1. DETAILED DESIGN TO INCLUDE RATIONALE AS TO HOW THERMAL MITIGATION OPTIONS DESCRIBED IN SECTIONS 2.8.6 AND 4.5 WERE CHOSEN OR NOT CHOSEN.
2. PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.

LEGEND

x(185.03)	PROPOSED SPOT ELEVATIONS
(TOP) 3:1 (BOTTOM)	EMBANKMENT (SLOPE AS NOTED)
	PROPOSED STORM SEWER
	FLOW DIRECTION
	MANHOLE
	HEADWALL(S)
	RIP RAP
	OVERLAND FLOW ROUTE

FIGURE 4.10 - CONCEPTUAL SWM POND

MTE
Engineers | Scientists | Surveyors

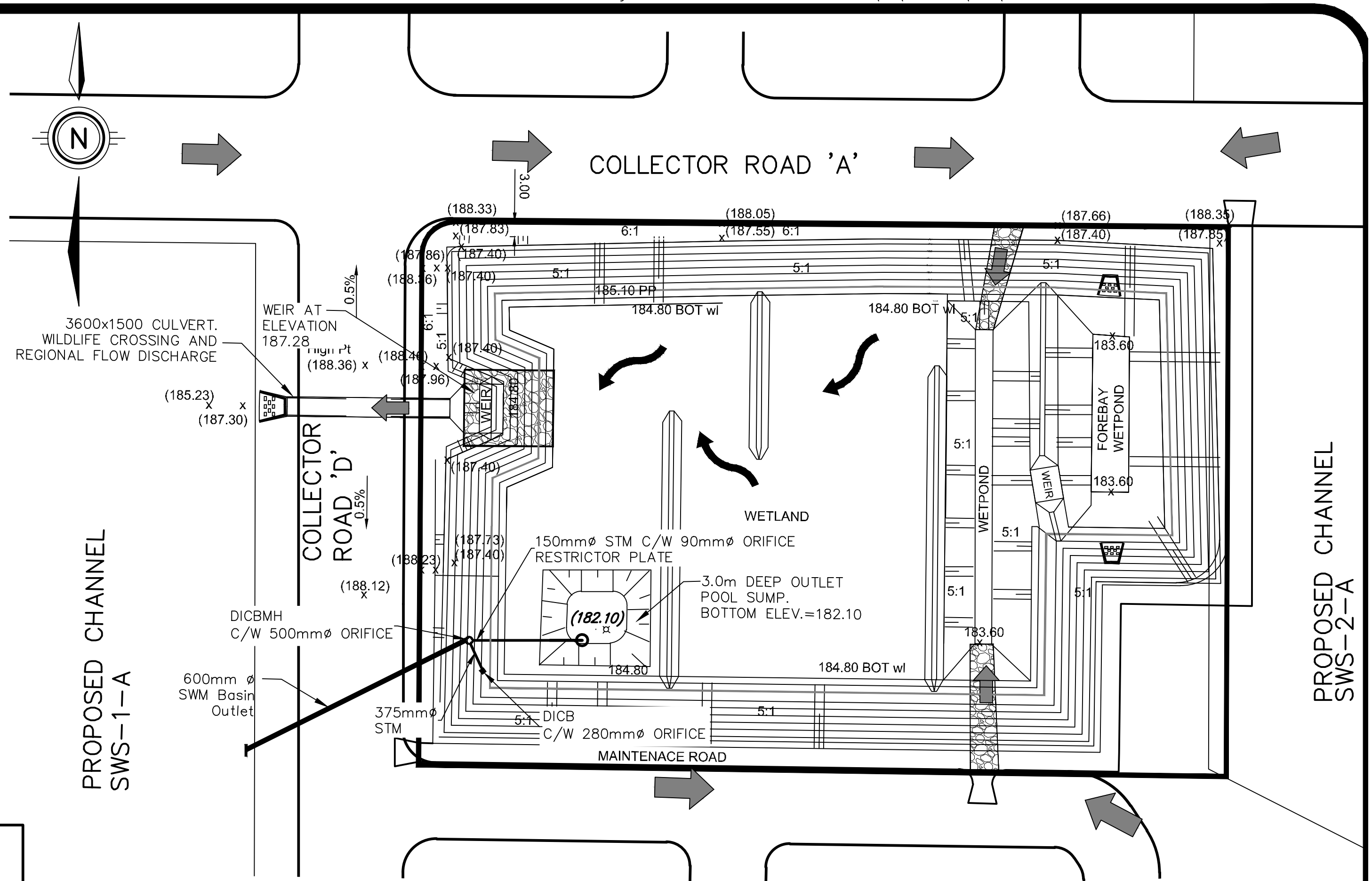
Project Name
SUBWATERSHED IMPACT STUDY - BLOCK 2

Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale.	MTE Project No.	Date	Layout No.
1:1500	10477-100	AUGUST 2015	M4-10-POND-J

NOTES:

1. DETAILED DESIGN TO INCLUDE RATIONALE AS TO HOW THERMAL MITIGATION OPTIONS DESCRIBED IN SECTIONS 2.8.6 AND 4.5 WERE CHOSEN OR NOT CHOSEN.
2. PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.



FACILITY CHARACTERISTICS

PERMANENT WATER LEVEL	185.10m
PERMANENT POOL VOLUME	2,885m³
EXTENDED DETENTION LEVEL	185.78m
EXTENDED DETENTION VOLUME	5616m³
100 YR WATER LEVEL	186.56m
100 YEAR WATER VOLUME	13,338m³
REGIONAL WATER LEVEL	187.56m
REGIONAL DETENTION VOLUME	25,623m³
BLOCK SIZE	1.70ha
LENGTH TO WIDTH RATIO	6.3 : 1

LEGEND

x(189.79)	PROPOSED SPOT ELEVATIONS
(TOP) 3:1 (BOTTOM)	EMBANKMENT (SLOPE AS NOTED)
	PROPOSED STORM SEWER
	FLOW DIRECTION
	MANHOLE
	HEADWALL(S)
	RIP RAP
	OVERLAND FLOW ROUTE

FIGURE 4.11 - CONCEPTUAL SWM POND K

MTE
Engineers | Scientists | Surveyors

Project Name
SUBWATERSHED IMPACT STUDY - BLOCK 2

Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

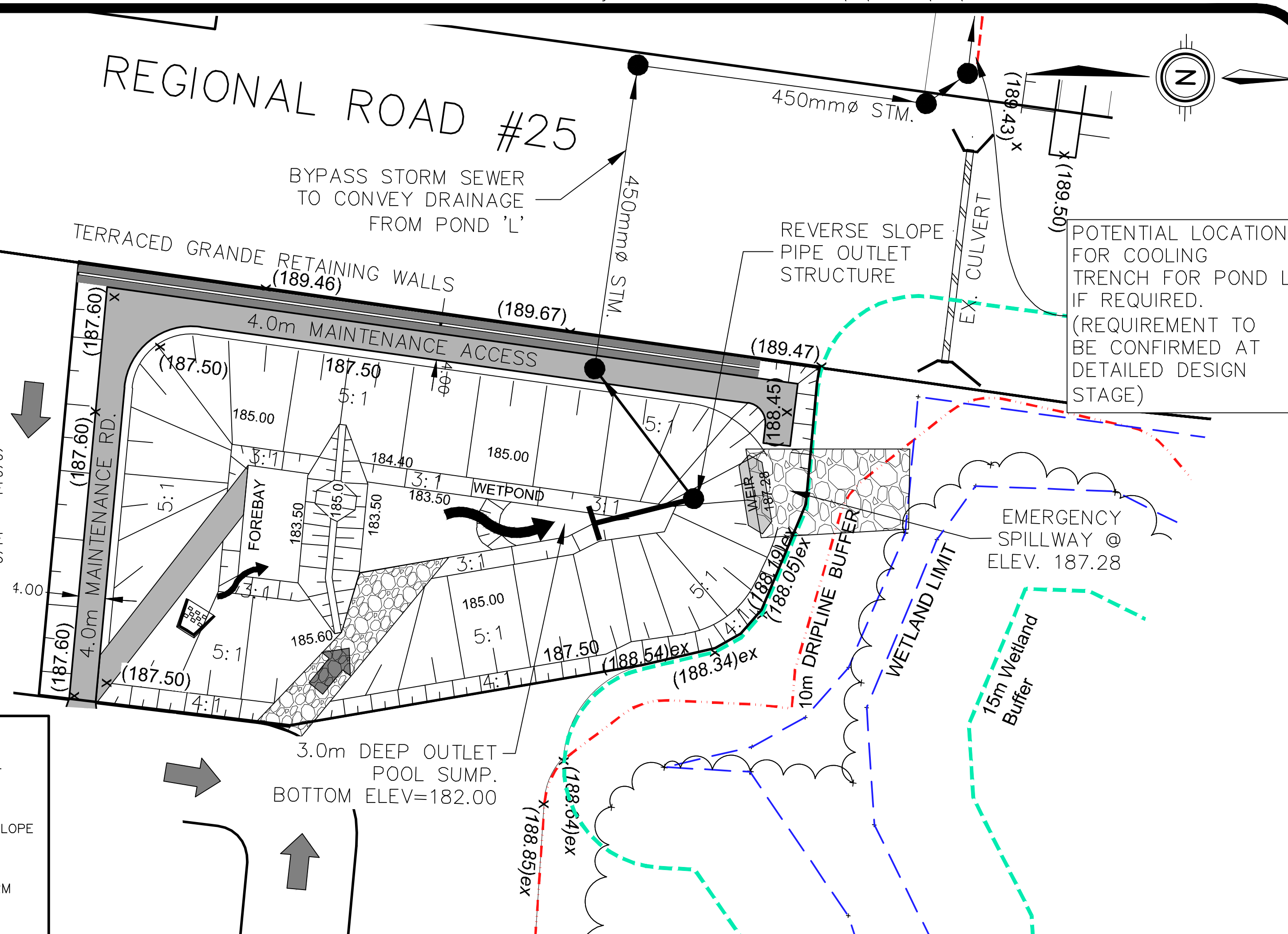
Scale.	MTE Project No.	Date	Layout No.
1:1500	10477-100	AUGUST 2015	M4-11-POND-K

FACILITY CHARACTERISTICS

PERMANENT WATER LEVEL	185.00m
PERMANENT POOL VOLUME	1,424m³
EXTENDED DETENTION LEVEL	185.88m
EXTENDED DETENTION VOLUME	1,680m³
100 YEAR WATER LEVEL	186.54m
100 YEAR WATER VOLUME	3,465m³
BLOCK SIZE	0.52ha
LENGTH TO WIDTH RATIO	3.3 : 1
REGIONAL DETENTION LEVEL	187.28m
REGIONAL DETENTION VOLUME	6,090m³

NOTES:

1. DETAILED DESIGN TO INCLUDE RATIONALE AS TO HOW THERMAL MITIGATION OPTIONS DESCRIBED IN SECTIONS 2.8.6 AND 4.5 WERE CHOSEN OR NOT CHOSEN.
2. PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.



LEGEND

x (185.03)	PROPOSED SPOT ELEVATIONS
(TOP) 3:1 (BOTTOM)	EMBANKMENT (SLOPE AS NOTED)
	PROPOSED STORM SEWER
	FLOW DIRECTION
	MANHOLE
	HEADWALL(S)
	RIP RAP
	OVERLAND FLOW ROUTE
	PERMANENT POOL
	FLOW PATH

FIGURE 4.12 - CONCEPTUAL SWM POND L

MTE
Engineers | Scientists | Surveyors

Project Name
SUBWATERSHED IMPACT STUDY - BLOCK 2

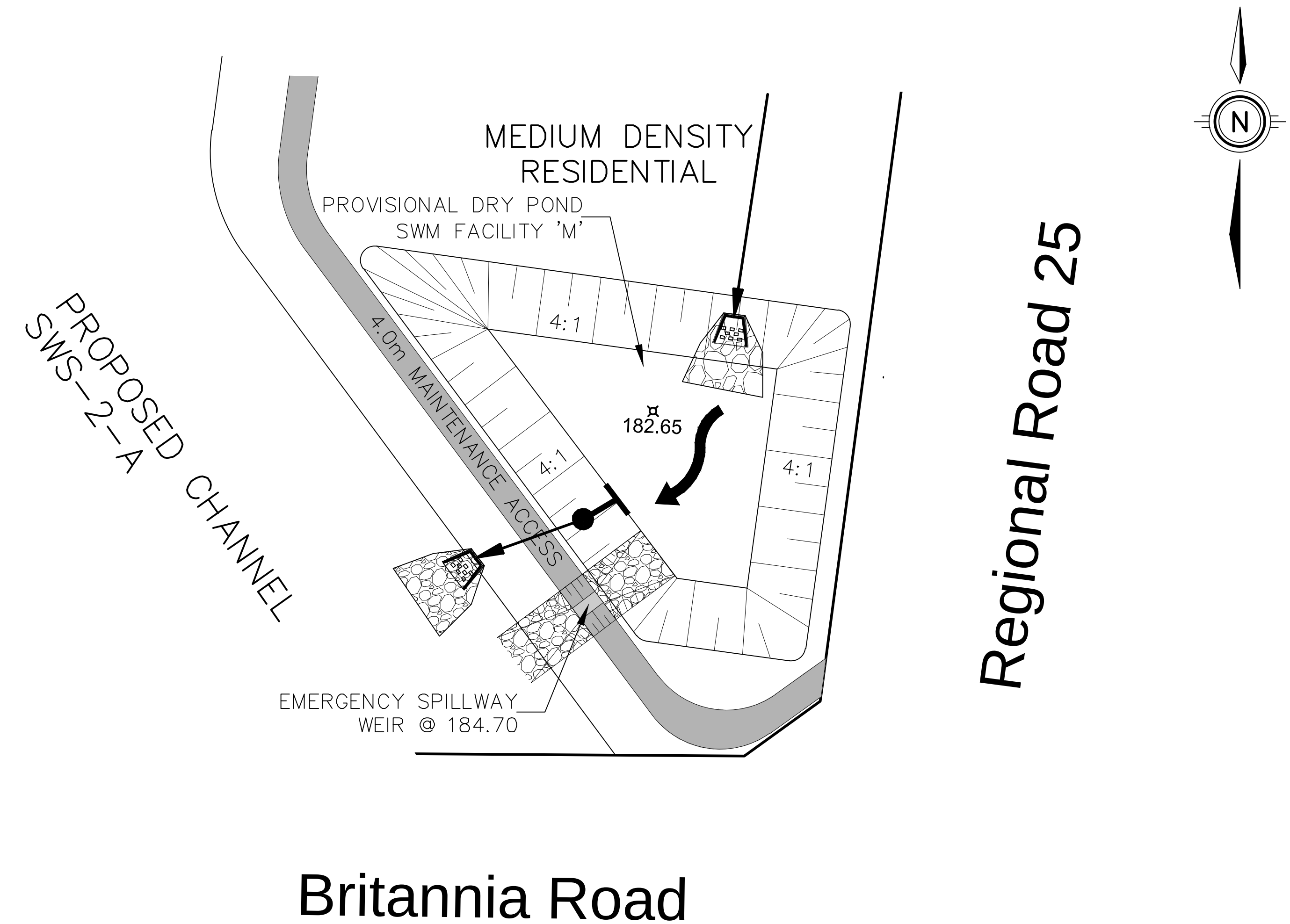
Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale.	MTE Project No.	Date	Layout No.
1:1000	10477-100	AUGUST 2015	M4-12-POND-L

August 25, 2015 — 10:41 a.m. — Plotted By: rmoir

Project: 10477–100 CAD: P:\P\10477\LD\10477–100–MS5–SWM PONDS.DWG

NOTE: PLANTINGS WITHIN SWM BLOCK TO BE IMPLEMENTED AS PER CONSERVATION HALTON'S LANDSCAPING AND TREE PLANTING GUIDELINES.



FACILITY CHARACTERISTICS	
POND BOTTOM ELEVATION	182.65
EXTENDED DETENTION LEVEL	183.37
EXTENDED DETENTION VOLUME	$672m^3$
100 YEAR WATER LEVEL	183.67
100 YEAR WATER VOLUME	$1,386m^3$
REGIONAL WATER LEVEL	184.70m
REGIONAL WATER VOLUME	$2436m^3$
BLOCK SIZE	0.42ha

LEGEND

x(189.79)	PROPOSED SPOT ELEVATIONS
(TOP) 3:1 (BOTTOM)	EMBANKMENT (SLOPE AS NOTED)
	PROPOSED STORM SEWER
	FLOW DIRECTION
	MANHOLE
	HEADWALL(S)
	RIP RAP
	OVERLAND FLOW ROUTE

FIGURE 4.13 - CONCEPTUAL PRO□ISIONAL SWM POND □M□

MTE

Engineers |Scientists |Surveyors

Project Name	
SUBWATERSHED IMPACT STUDY - BLOCK 2	
Site	Client
BOYNE SURVEY SECONDARY PLAN	BOYNE SURVEY LAND OWNERS GROUP

Scale.	MTE Project No.	Date	Layout No.
1:750	10477-100	AUGUST 2015	M4-13-POND-M

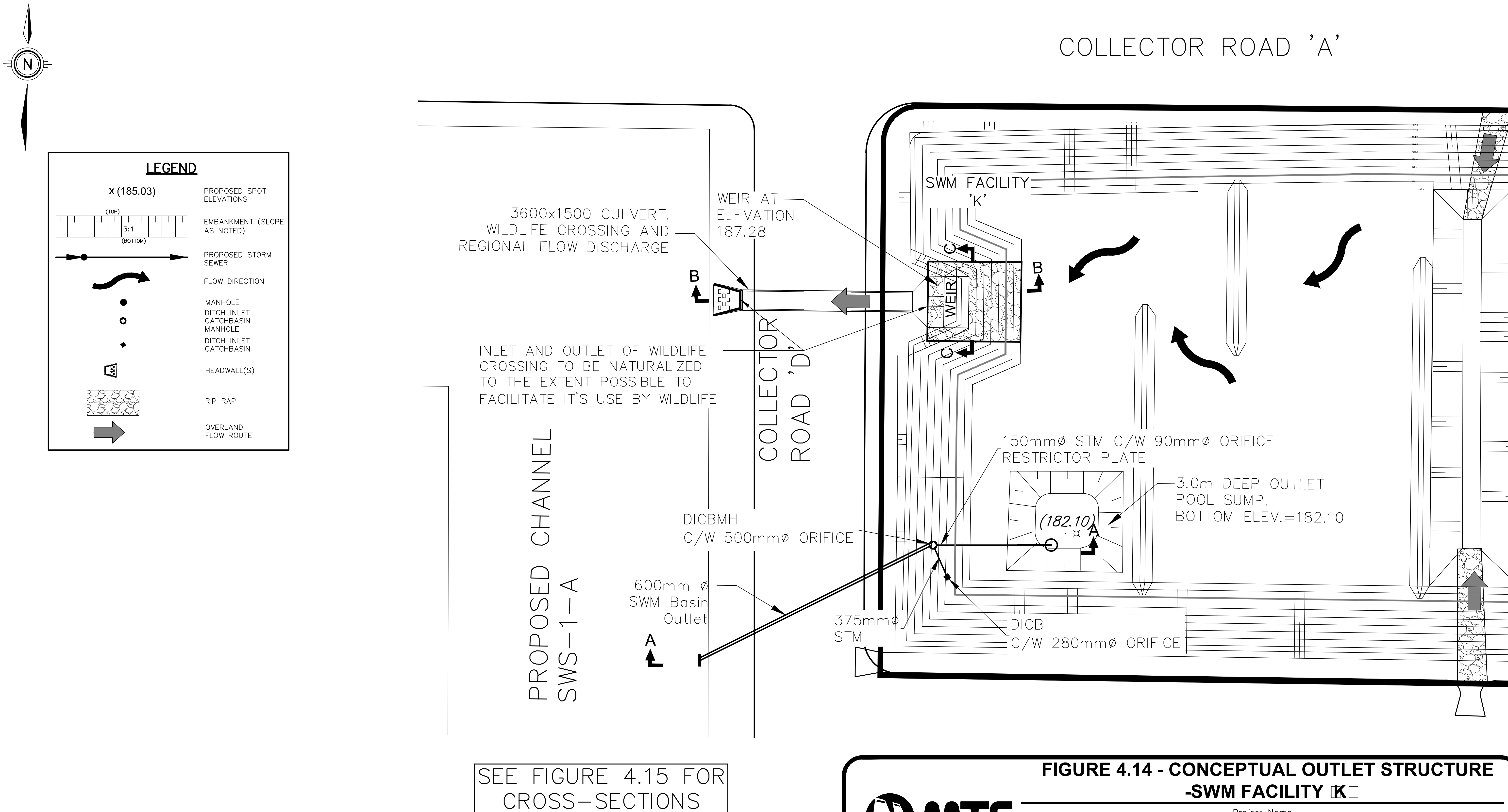


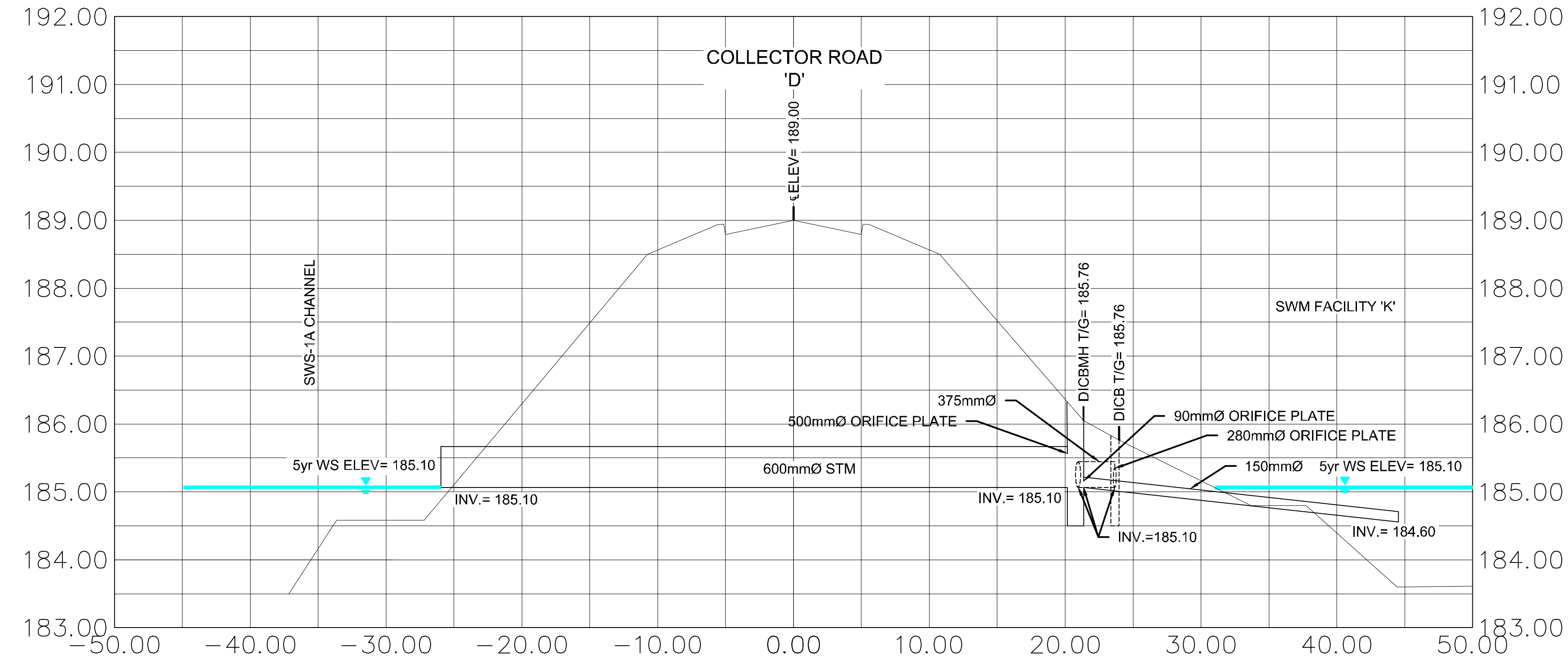
FIGURE 4.14 - CONCEPTUAL OUTLET STRUCTURE -SWM FACILITY 'K'

MTE
Engineers | Scientists | Surveyors

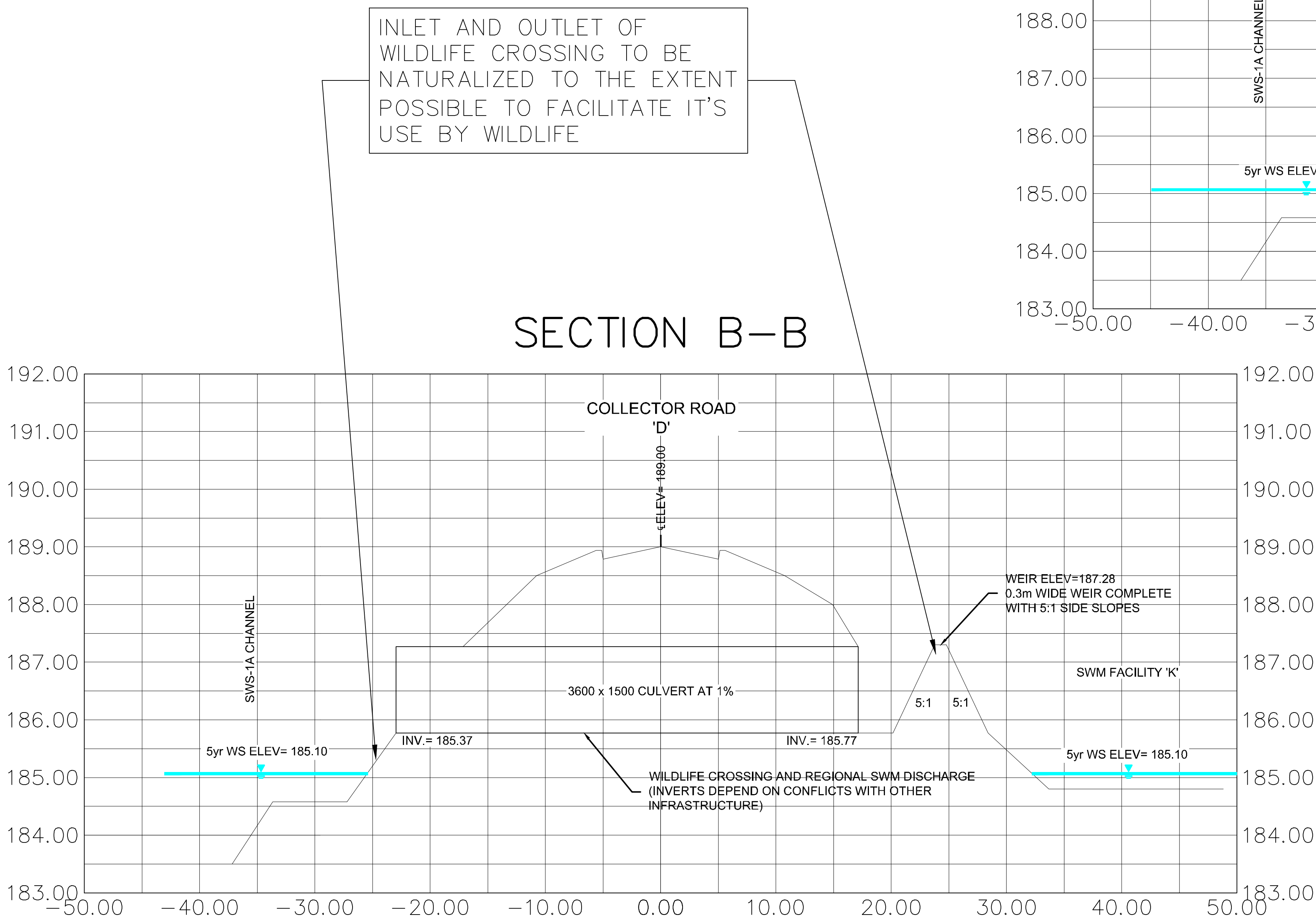
Project Name			
SUBWATERSHED IMPACT STUDY - BLOCK 2			
Site		Client	
BOYNE SURVEY SECONDARY PLAN		BOYNE SURVEY LAND OWNERS GROUP	
Scale.	MTE Project No.	Date	Layout No.
1:750	10477-100	AUGUST 2015	M4-14-POND-K

August 25, 2015 10:42 a.m. Plotted By: rmoir

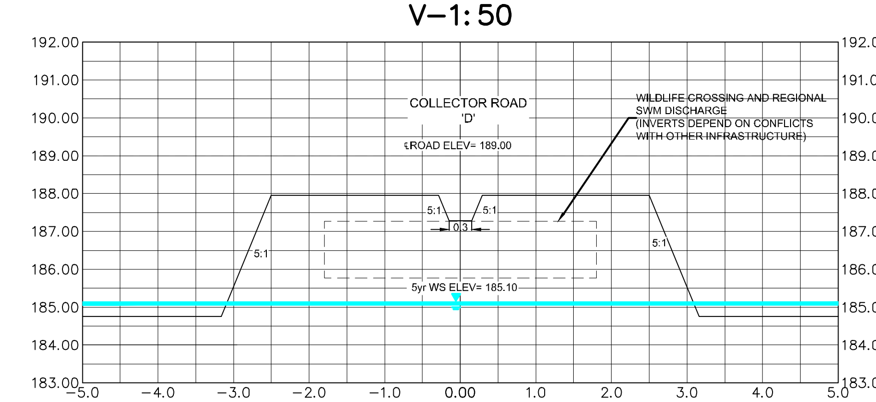
SECTION A–A



SECTION B–B



SECTION C–C
H–1:100
V–1:50

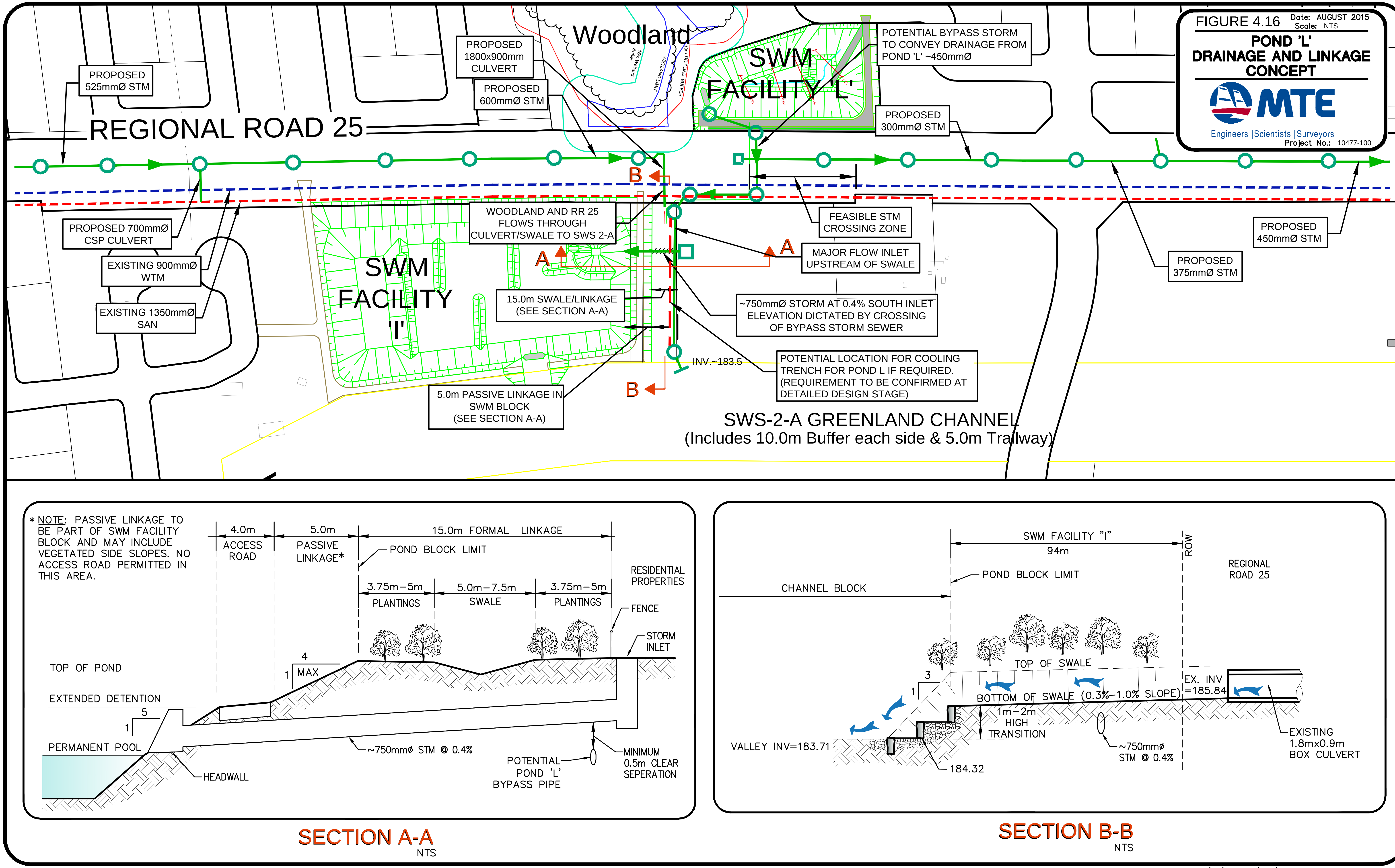


MTE
Engineers | Scientists | Surveyors

FIGURE 4.15 - CONCEPTUAL OUTLET STRUCTURE -SWM FACILITY 'K' CROSS SECTIONS

Project Name			
SUBWATERSHED IMPACT STUDY - BLOCK 2			
Site: BOYNE SURVEY SECONDARY PLAN		Client: BOYNE SURVEY LAND OWNERS GROUP	
Scale.	MTE Project No.	Date	Layout No.
H-1:500 V-1:100	10477-100	AUGUST 2015	M4-15-POND-K

August 25, 2015 10:46 a.m. Plotted By: rmoir



August 25, 2015 — 10:59 a.m. — Plotted By: rmoir

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4.6 Open Channel Design

Tributaries SWS-1-A and SWS-2-A are proposed to be realigned as part of the development of the Study Area. These systems will be contained within buffered watercourse corridors, which will traverse the site from north to south as illustrated on Drawings 4.1, 4.2, and 4.3. The upstream and downstream tie-in points for each system are existing culverts and channels. The horizontal and vertical locations of the proposed channels are controlled at the upstream end by the existing Louis St. Laurent culverts, and at the downstream end by the existing tributary elevations at Britannia Road.

The conceptual design of the proposed channel systems has considered fluvial geomorphology, riparian flood storage volumes, flood conveyance with provision of appropriate freeboard, and watercourse buffers. The preliminary valley section bottom widths were selected based on the greater of the required meander belt (including safety factor), the minimum bottom width required to provide appropriate riparian storage volumes, and 20m (as set out by the FSEMS Implementation Principles).

Valley bottom widths established for SWS-1-A and SWS-2-A are listed in Table 4.7.

TABLE 4.7 – SWS-1-A AND SWS-2-A VALLEY BOTTOM WIDTHS

Channel	Reach	Final Meander Belt Width including Factor of Safety[1] (m)	Minimum Meander Belt Width per Implementation Principles (m)	Channel Valley Bottom Width included in HEC-RAS model (m)
SWS-1-A	Louis St. Laurent Boulevard to Station 1250	12.6	20	20.50
	Station 1250 to Station 820	16.3	20	22.40
	Station 820 to Britannia Road	23.4	20	25.45
SWS-2-A	Louis St. Laurent Boulevard to Station 868	17.9	20	22.20
	Mid-Block 2 (Station 868)	20.5	20	25.90
	Station 868 to Britannia Road	24.0	20	25.90

[1] As per “Meander Belt Width Analysis Indian Creek Tributaries I-NE-2A & I-NE-1B Sixteen Mile Creek Tributaries SWS-1-A & SWS-2-A Boyne Survey Town of Milton” (Aqualogic Consulting, June 24, 2013).

The selected depths within all reaches of each tributary provide a minimum 0.3 m freeboard between the Regional storm floodline elevation and the outside edge of the channel corridor (outer limit of buffers). Further discussion on the channel design and hydraulic analyses is provided in the following sections.

Proposed Channel Design

A conceptual natural channel design has been undertaken by Aqualogic Consulting to determine at a preliminary level the appropriate geomorphic form and function for the tributaries to be re-aligned. The results of this assessment are presented in Appendix A8.

The conceptual design of the system reflects run-pool and run-channel features mixed with linear wetlands and wet meadows. The intent is to redefine the existing altered stream form to improve hydrologic function and provide potential fish habitat features, while realizing an overall stable geomorphic form. Each tributary has been divided into two reaches, namely, the upper reach which extends from Louis St. Laurent Avenue at the north to mid-block and the lower reach which extends from mid-block to Britannia Road. The south reaches will have relatively low gradients, near 0.22%, while the upper reaches have valley gradients of 0.82 – 0.99%.

The following tables along with the detailed supporting documentation in Appendix A8 summarize the proposed typical channel configuration and design cross-sections for each reach.

TABLE 4.8 – CONCEPTUAL CHANNEL DESIGN PARAMETERS– SWS-1-A

	Upper Reach	Lower Reach
Valley gradient (%)	0.99	0.22
Valley depth (m)	0.90 – 1.27	1.28 – 1.59
Valley Bottom Width (m)	20.5 – 22.4	25.45
Meander belt inc. safety factor (m)	<20	23.4
Low-flow channel gradient (%)	0.91	0.20
Bankfull top width (m)	3.5	4.0

TABLE 4.9 – CONCEPTUAL CHANNEL DESIGN PARAMETERS – SWS-2-A

	Upper Reach	Lower Reach
Valley gradient (%)	0.82	0.22
Valley depth (m)	1.22 – 1.54	1.52-1.82
Valley Bottom Width (m)	22.2	25.9
Meander belt inc. safety factor (m)	<20	24
Low-flow channel gradient (%)	0.75	0.21
Bankfull top width (m)	3.5	4.0

Watercourse Buffers

A 10 metre watercourse buffer has been applied for watercourses SWS-1-A and SWS-2-A. On the west side of each drainage feature, a 5 m community trail has also been incorporated into the channel corridor. Typical preliminary cross-sections and corridor dimensions are illustrated on Drawings 4.2 and 4.3.

Regional Storm Control and Hydrologic Verification

The current version of the FSEMS (AMEC, May 2015) includes a requirement for Regional Storm peak flow control to existing conditions rates within all areas of the Boyne Survey. Existing conditions peak flow targets at the Britannia Road nodes are included within the FSEMS, and updated values based on optimization of the stormwater management facilities for 2 – 100 year and Regional storm control were provided by AMEC subsequent to FSEMS publication (refer to June 15/15 correspondence included in Appendix C2). For SWS-1-A the existing conditions rate is 9.84 m^3/s, and for SWS-2-A it is 18.2 m^3/s. The FSEMS concluded that providing the requisite Regional Storm peak flow control within end-of-pipe facilities would result in an approximate 100% increase in the volume of storage required in the SWM facilities. Within the FSEMS Implementation Principles, on-line storage for Regional Storm control was approved in principle subject to demonstration that the proposed approach addresses fluvial geomorphologic requirements, provides for fish and wildlife passage of target species, and provides thermal impact mitigation.

The proposed Regional Storm Control Strategy recommended in this report evolved through consultation with the Town of Milton and Conservation Halton. The draft version of the Block 2 SIS Submitted on March 28, 2014 included a combination of off-line and on-line control of the Regional Storm in accordance with the 2013 FSEMS. This “hybrid” solution included two online control structures in each channel, which backed water up into the SWM ponds during the Regional Storm.

The hydrologic verification completed by AMEC concluded that the proposed hybrid solution provided the required Regional Storm control. However, Conservation Halton expressed concern with the on-line control strategy. Through consultation with the Town and AMEC a modified “hybrid” solution, was developed with only one on-line control structure in each channel. The hydrologic verification completed by AMEC concluded that this hybrid solution also provided the required Regional Storm control. However, Conservation Halton continued to express concern about the ecological impacts of the proposed control structures.

AMEC completed a further hydrologic verification of the 100% Off-line Regional Storm Control strategy. This was based on providing Regional Storm storage in the SWM ponds above the 100 year flood elevation. The analysis concluded that the SWM pond footprints had to be increased in size by approximately 15% to 20% on average to accommodate the additional storage. On that basis, the required Regional Storm control was provided off-line.

In order to expedite the completion and approval of the Block 2 SIS, the Block 2 Land Owners decided to proceed with 100% Off-line Control for the Regional Storm event, for the following reasons:

1. The Town has agreed to allow Regional Storm control storage within the SWM blocks above the normal maximum 1.8m depth allowed for the 100 Year Storm;

2. Based on the hydrologic verifications and SWM pond optimization analyses completed by AMEC, the Unitary Storage volumes per Impervious Hectare have reduced from 2100 and 1950 m^3/imp. ha to 1825 and 1450 m^3/imp. ha, for SWS-1-A and SWS-2-A respectively. This has reduced the size of the SWM block required for 100% off-line control;
3. Without on-line control, the Regional flood elevation in the channels is lower, which allows the channel widths to be reduced, which helps to offset the additional land required for the SWM blocks; and
4. CH continues to oppose on-line controls.

As such, 100% offline Regional flood storage is provided in each proposed SWM facility within Block 2.

Hydraulic Evaluation

Hydraulic analyses to determine post-development floodplain limits, regional and riparian storage for the SWS-1-A and SWS-2-A tributaries were conducted using the HEC-RAS hydraulic modeling software. The location of the proposed condition model cross-sections are shown on Drawings 4.2 and 4.3. Available modeling upstream and downstream of the Boyne Survey has been incorporated into the Block 2 model. Modeling of downstream reaches to the confluence of SWS-1-A and SWS-4-A approximately 1km south of Britannia Road (i.e. 450m downstream of the confluence of SWS-1-A and SWS-2-A) was prepared by MTE. Flow values for each design storm at locations along the two corridors were provided by AMEC from the upper limits of the model through to the lower limits.

Boundary conditions were included in the model based on SWS-2-D modeling obtained from David Schaeffer Engineering Ltd. The SWS-2-D crossing of Louis St. Laurent Avenue was based on best available as-constructed information obtained from the Town of Milton.

As previously noted, appropriate freeboard of at least 0.3 m is maintained between the outer limits of the channel buffer and the Regional storm water surface elevation.

As previously noted, the existing conditions hydraulic modeling for the Study Area reaches was provided by AMEC. This model, combined with updated (i.e. 2015) flow data received from AMEC was utilized to determine existing conditions riparian storage volumes for the SWS-1-A and SWS-2-A reaches.

The details of the regional and riparian storage and backwater calculations for the post-development conditions for SWS-1-A and SWS-2-A are included in Appendix C6. Tables 4.10 and 4.11 provide summaries of the riparian storage analyses for the respective tributaries. The HSP-F hydrologic modeling results from the FSEMS (return period peak flow rates) have been used for the preliminary regional and riparian storage analysis (see flow summary in Appendix C6).

TABLE 4.10 – RIPARIAN STORAGE ANALYSIS RESULTS – SWS-1-A

Event	Existing Storage (x 1,000 m^3)	Proposed Storage (x 1,000 m^3)
Regional Storm	23.81	25.40
100-year	10.87	12.24
50-year	9.51	10.71
20-year	7.90	8.75
10-year	6.64	7.34
5-year	5.26	5.89

TABLE 4.11 – RIPARIAN STORAGE ANALYSIS RESULTS – SWS-2-A

Event	Existing Storage (x 1,000 m^3)	Proposed Storage (x 1,000 m^3)
Regional Storm	36.30	40.05
100-year	13.66	17.03
50-year	11.81	14.86
20-year	9.51	12.10
10-year	7.87	10.04
5-year	6.36	8.22

It should be noted that the bankfull channels within the SWS-1-A and SWS-2-A valleys meander with a sinuosity factor of 1.1, meaning the length of the meandering bankfull channel is 10% greater than the length of the main valley. While additional riparian storage is used in this extra length of bankfull channel, it is not represented in the HEC-RAS model. To account for this extra storage used under each design storm event, the Proposed Storage values listed in Tables 4.9 and 4.10 have been adjusted by 140-160 m^3 and 170 m^3 in channels SWS-1-A and SWS-2-A, respectively.

Moreover, the HEC-RAS model does not consider the additional storage volume provided in scour pools, pocket wetlands and other depressions in the channel valley. Given typical dimensions of the wetland features of 0.5 m in depth, 5-10 m in width, 10-15 m in length, and 100m spacing, the Proposed Storage values listed in Tables 4.10 and 4.11 above have been further adjusted by 750 m^3 and 800 m^3 for SWS-1-A and SWS-2-A respectively, to conservatively account for depression storage in the channels.

From Table 4.10 and 4.11, it can be seen that the criterion to meet or exceed pre-development riparian storage in the post-development scenario has been met in all cases.

Watercourse Crossings

As shown on Drawings 4.1-4.3, the proposed internal road network within the Boyne Survey including Britannia Road and Regional Road 25 will require the construction or reconstruction of several watercourse crossings.

Section 4.3 of this report summarized the general requirements applicable to the sizing of the watercourse crossings within the Study Area. In addition, for major events (1:100 and Regional) Town of Milton Engineering Standards require that transverse water crossings shall have a maximum depth at the crown of the road of 0.15 m and a maximum velocity of 0.4 m/s. Arterial road crossings are to be sized for the 1:100 year to Regional storm, collector roads for the 1:50 year storm, and urban local roads for a 1:25 year storm.

Note that conveyance of the design storm event is a minimum conveyance requirement for the crossing. The culvert size was further refined by the desire to maintain appropriate freeboard within the channel corridor (0.3 m minimum from outer edge of buffer) during the Regional storm event considering the inclusion of the road crossings. The proposed conditions hydraulic model was utilized to determine a recommended structure size for appropriate conveyance of flows.

As discussed within the Natural Channel and Wetland Corridor Preliminary Design (Aqualogic) included within Appendix A8, the recommended minimum crossing span is to provide a 1 m setback on either side of the bankfull width. However, the Town of Milton and Conservation Halton have expressed a strong desire for crossing widths of three times the bankfull width. Hence, the crossing widths provided are equal to three times the bankfull width. The recommended bankfull widths within the upper and lower reaches are 3.5 m and 4.0 m respectively for the preliminary design of both SWS-1-A and SWS-2-A. As such minimum spans of 10.5m for the upper reaches and 12.0 m for the lower reaches have been utilized.

Note that two culverts within the Study Area have recently been replaced as part of the Regional Road 25 road reconstruction. Crossing 'F' now consists of twin 2.4 x 1.2m concrete box culverts. The mid-block crossing of Regional Road 25 (Crossing 'G' near proposed SWM facilities 'L' and 'I') was also replaced and consists of a 1.8 x 0.9 m box culvert.

Drawing 4.1 illustrates locations of all proposed crossing sizes within the study limits, which have been sized based on the above noted considerations and summarized in Table 4.12.

Note that the Britannia Road crossing of SWS-1-A is being addressed within the Britannia Road Transportation Corridor Municipal Class Environmental Assessment process, which is currently in progress. The crossing design is summarized within the report completed as part of that process, which is entitled "Technical Report – Hydraulic Analyses of Stream Crossings and Stormwater Management Alternatives Assessment" (Aquafor Beech, draft). The structure size recommended within that report is reflected within Table 4.12 for reference. The size will need to be confirmed as part of the final design of Britannia Road, taking into account the preliminary analyses included within this report.

TABLE 4.12 – CONCEPTUAL DESIGN OF PROPOSED ROAD CROSSINGS

Culvert Crossing ID	Watercourse	Location	Conceptual Culvert Size (Span x Avg. Opening Height (m))*	Conceptual Culvert Size Currently Proposed (Span x Avg. Opening Height (m))
A	SWS-1-A	Internal Collector	5.5 x 1.2	10.5 x 1.2
B	SWS-1-A	Internal Collector	6.1 x 1.4	12 x 1.5
C*	SWS-1-A	Britannia Road	7.6 x 1.5	7.6 x 1.5
D	SWS-2-A	Internal Collector	5.5 x 1.3	10.5 x 1.2
E	SWS-2-A	Internal Collector	6.1 x 1.9	12 x 1.8
F*	SWS-2-A	Britannia Road	10.2 x 1.2	10.2 x 1.2

*Size as per the preliminary recommendations from the Britannia Road Class EA Technical Report – Hydraulic Analyses of Stream Crossings and Stormwater Management Alternatives Assessment (Aquafor Beech, draft), based on correspondence with Class EA team.

It is envisioned that the internal road crossings (A, B, D, and E) will consist of open-bottom precast concrete structures. It is anticipated that crossing C (SWS-1-A at Britannia Road) and F (SWS-2-A at Britannia Road) will be constructed by Halton Region as part of the urbanization of Britannia Road.

Overall Channel Corridor Widths

Following completion of the channel assessments described in the several preceding subsections, overall channel corridor widths for SWS-1-A and SWS-2-A were established in the following manner:

a) The minimum channel valley bottom width was established as the greater of the meander belt width including relevant factor of safety, or the minimum of 20m as set out by the FSEMS Implementation Principles.
b) 3:1 side slopes up from the outer limits of the valley bottom were added.
c) The Regional storm water surface elevation at each HEC-RAS river station was determined by routing the Regional storm flows through the channels, to establish the hazard area.
d) Outside of the hazard area, a 10m buffer was added to one side of the channel, and a 15m buffer was added to the other to accommodate a walkway and/or Multi-Use Trail.

e) Riparian storage was modeled for the post-development scenario and compared to the pre-development scenario to ensure that this is not a limiting factor to the channel corridor widths.

Beginning with the channel valley bottom widths listed in Table 4.7, and applying the methodology described above, minimum channel corridor widths were determined to range from 48.22 m to 59.68 m on channel SWS-1-A, and from 51.23 m to 58.48 m on channel SWS-2-A. In the interest of simplifying the geometry of the channel corridors, the widths have been 'normalized' to range from 49.0 m to 56.7 m on SWS-1-A, and from 52.1 m to 58.5 m on SWS-2-A. The normalized channel corridor widths are illustrated on Drawings 4.2 and 4.3. Appendix C7 includes channel corridor calculation data and results.

4.7 Preliminary SWM Measure Operation and Maintenance Recommendations

Stormwater management facilities will require periodic maintenance to sustain long term effectiveness for pollutant removal and water quantity control. It is recommended that a monitoring and maintenance program be developed as part of the detailed design for each facility, to help ensure its long term effectiveness. The post-construction monitoring program that is included within Section 6.0 of this report includes recommendations for performance evaluation monitoring.

With respect to the individual SWM facilities, at the time of detailed design, an operation and maintenance plan should be prepared, which lists all of the components of the facility. The plan should describe the function and operation of each component and most importantly recommend what maintenance will be required to the component to keep the facility operating efficiently.

It is recommended that during construction of the SWM facility, monitoring and inspection of the erosion and sediment controls be conducted to ensure the satisfactory performance of these measures.

Furthermore, it is recommended that the Town of Milton, as the owner of the facility, initiate a post-construction monitoring program to ensure the long term effectiveness of the SWM facility. The recommended components of the monitoring program are as follows:

1. Detailed visual inspection of all major components of the SWM facility on a regular (seasonal) basis. The intent of this type of inspection is to identify potential problems before they occur (such as partial or full outlet blockage, erosion around key structures, sediment build up, etc.), and as such should be considered a thorough review. Remedial action should be taken by the Town as recommended / required.
2. Inspection of the SWM facility after significant rain events. This inspection is to include a relative observation of water level within the facility and ensuring that

the outlet is operating. The presence or absence of any damage to key components of the facility should be noted.

3. Review of sediment accumulation within the facility. Sediment build up should be visually inspected on a regular basis in association with Task 1, and sediment surveys (bathymetric measurements of the forebay) completed every 5-10 years. Once sediment accumulation in the sediment forebay has reached one half of the depth of the sediment forebay, it is recommended that the facility be drained and the sediment be vacuum excavated for transport to a suitable disposal facility.

Documentation of all monitoring/inspection/maintenance activities is to be maintained in a log book, referencing all inspection reports.

The on-site control measures implemented within areas 407 and 411 may also include on-site detention measures (e.g. rooftop control devices, parking lot storage, subsurface storage) with flows limited by hydraulic control devices (e.g. orifice). Water quality treatment units (e.g. oil-grit separators) will also likely be implemented. The form of these measures is to be determined through final design of these site plans. The documentation accompanying these plans should include recommended operational and monitoring practices such as regular orifice inspection, and frequency of water quality treatment unit cleanout.

4.8 Erosion and Sediment Control Measures

During the detailed design stage of development, and prior to initiation of area grading and servicing, erosion and sedimentation control plans should be prepared. The plans must conform to the Town of Milton and Conservation Halton criteria and illustrate the erosion and sediment control measures to be implemented during construction, which will limit the impacts associated with development.

The Erosion and Sedimentation Control strategy to be implemented during the detailed design stage should include the following aspects:

i. Priority focus on erosion control (proactive) versus sediment control (reactive);
ii. Strategic and careful consideration of land clearing and phasing requirements such that vegetated areas are not disturbed unnecessarily or for extended periods;
iii. ESC plans should be designed, implemented and monitored by qualified personnel (i.e. Certified Inspector of Sediment and Erosion Control (CISEC), Certified Professional in Erosion and Sediment Control (CPESC) or suitable equivalent); and
iv. ESC plans should be completed for each phase of development where drainage patterns are modified, namely the earthworks, servicing and construction/homebuilding phases.

Typically, the recommended sequence for the implementation of erosion and sediment control measures should be as follows:

- Installation of all required temporary sediment control fencing prior to the commencement of grading works, including 7.5m from the edge of channels.
- Temporary vehicle tracking controls installed and maintained at all access points.
- Construction of the permanent and temporary stormwater management ponds that will serve as sedimentation basins for each development area during construction.
- Construction of temporary swales to direct runoff to sedimentation basins, with rock check dams as required to control velocities.
- Removal of vegetation in accordance with all applicable by-laws.
- Stripping and strategic placement of topsoil stockpiles. Placement of sediment control fencing around all stockpile areas.
- Temporary seeding of topsoil stockpiles where stockpiles are expected to remain undisturbed for an extended period of time.
- Inspection and maintenance by the contractor of temporary erosion and sediment control measures during construction (including periodic cleaning as required) until such time that the Engineer or Town of Milton approves their removal.
- Side slopes of all temporary diversion channels are to be seeded with a fast growing nurse crop or stabilized through the placement of sod prior to the channel receiving flows.
- Re-vegetation of completed areas as soon as possible after construction.

5.0 GRADING AND MUNICIPAL SERVICES

5.1 Site Grading Design

The proposed grades for Block 2 are primarily governed by the overall drainage scheme for the site and a balanced cut/fill design. The site is drained by two (2) existing watercourses running from north to south, SWS-1-A and SWS-2-A. These watercourses are proposed to be realigned and lowered to accommodate efficient land use on the site, and will be reconstructed in accordance with natural channel design principles. The channels will be kept as flat as possible in order to keep the storm sewer outlets as low as possible to minimize the amount of fill required in the southerly half of the site.

Other significant criteria governing the grading of the site are as follows:

- The invert of outlet pipes from the SWM facilities and outlet structure design should be completed considering potential backwater effects from the more frequently occurring water levels within the channels (e.g. events with a 5-year return period or less).
- The road right-of-way adjacent to the SWM facilities must maintain a 0.3m freeboard above the regional flood elevation.
- Overland flow routes must be provided to convey runoff in excess of the storm sewer system capacity to the SWM facilities.

A-2 Excerpts from TMIG SWM Pond "I" Report (Sept 2016)

Milton Phase 3, Boyne Survey Area, Block 2

Gulfbeck Developments Subdivision – Stormwater Management Design Report – SWM Pond I

TOWN OF MILTON ▪ SEPTEMBER 2016

REPORT PREPARED FOR

Gulfbeck Developments Inc.
3751 Victoria Park Avenue
Toronto, ON M1W 3Z4

REPORT PREPARED BY

THE MUNICIPAL INFRASTRUCTURE GROUP LTD.
8800 DUFFERIN STREET, SUITE 200
VAUGHAN, ON L4K 0C5
(905) 738-5700

TMIG PROJECT NUMBER 13138

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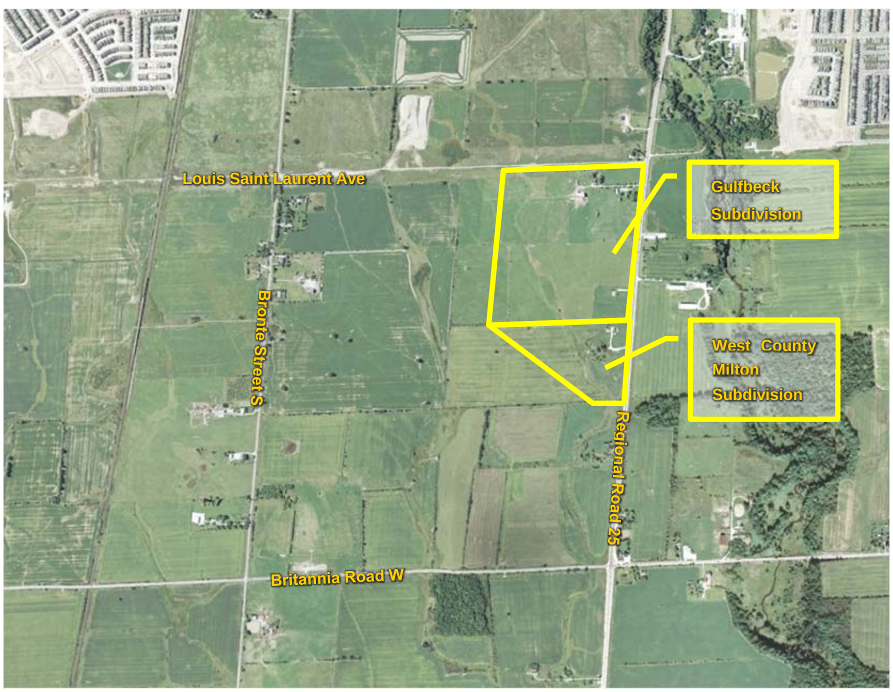
1 INTRODUCTION

1.1 Objective

This report is provided in support of the stormwater management design for the proposed Gulfbeck Developments Inc. Subdivision and the North-East West County Milton Properties Ltd. Subdivision in the Town of Milton. The Gulfbeck and West Country Milton Subdivisions are tributary to stormwater management facility, SWM Pond ‘I’, which is entirely within the West Country Milton Subdivision (lands south of Gulfbeck). As illustrated in **Figure 1-1**, the Gulfbeck and West Country Milton Subdivisions are located south of Louis Saint Laurent Avenue and west of Regional Road 25, within Block 2 of the Boyne Survey Area, in the Town of Milton.

The objective of this report is to demonstrate that the storm sewer system from the subject site and the stormwater management facility, SWM Pond ‘I’, have been designed following the recommendations set out in the Functional Stormwater and Environmental Management Strategy (FSEMS), prepared by AMEC, dated November 2015 and the Boyne Survey Block 2 Subwatershed Impact Study (SIS), prepared by MTE, dated July 2015. This report demonstrates that SWM Pond ‘I’ will provide the appropriate water quality treatment and water quantity attenuation such that the applicable criteria are satisfied. The detailed design drawing set should be referenced in conjunction with the review of this report. Copies of the SWM Pond drawings are provided in **Appendix D**.

Figure 1-1: Site Location



1.2 Background Reports

The following reports have been compiled historically for the subject site:

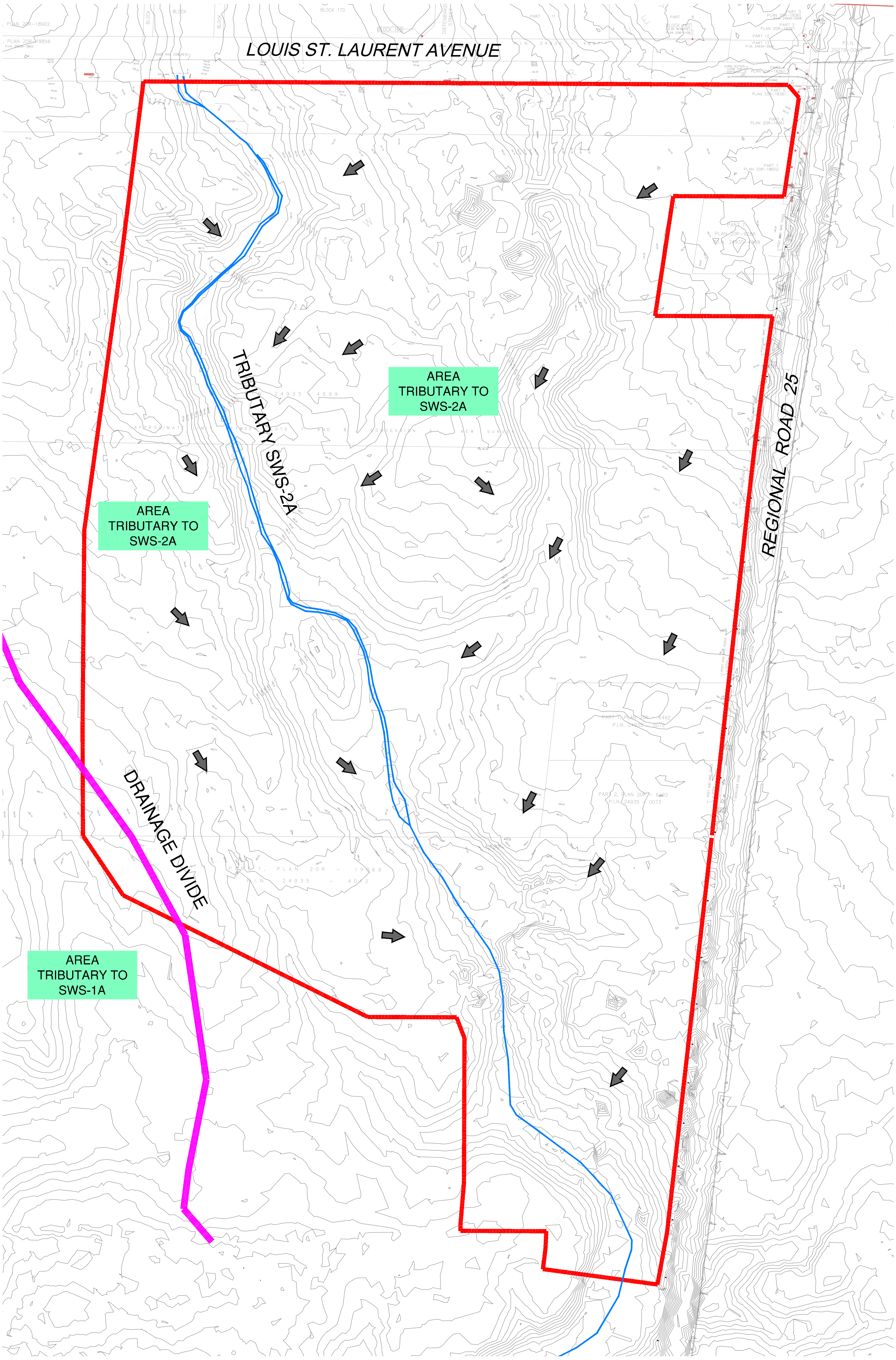
- Sixteen Mile Creek, Areas 2 and 7 Subwatershed Update Study (SUS), November 2015;
- Functional Stormwater and Environmental Management Strategy (FSEMS), Boyne Survey Secondary Plan Area, Final, November 2015, including the Implementation Principles for the Boyne Survey Natural Heritage System (Appendix I);
- Boyne Survey Block 2, Subwatershed Impact Study (SIS), Town of Milton, July 2015;
- Updated Hydrogeological Assessment, Gulfbeck Subdivision, Milton, July 2015, prepared by R.J. Burnside & Associates Limited;
- Gulfbeck Post-Development Water Balance, Milton, August 2016, prepared by R.J. Burnside & Associates Limited;
- A Soil Investigation for Proposed Residential Development, Part of NW ½ of NE ½ Lot 7 and Part of SE ½ of NE ½ Lot 8, Concession 2 New Survey, Town of Milton, January 2004, prepared by Soil-Eng Limited; and
- Geotechnical Letter, Proposed Stormwater Management Facility – Pond 'I', Country Homes Milton Phase 3 Lands, Regional Road 25, between Louis St. Laurent Avenue and Britannia Road, Town of Milton, August 2016, prepared by Soil-Eng Limited.

1.3 Existing Conditions

The study area is comprised mainly of agricultural lands, with a relatively flat topography. The topography slopes from an elevation of 193.0 m at Louis St. Laurent Avenue to 187.5 m at the proposed location for SWM Pond 'I', in a south to south-west direction towards the existing watercourse (tributary SWS-2-A). The existing drainage is shown on **Figure 1-2**.

The study area is located in the Sixteen Mile Creek subwatershed. Tributary SWS-2-A, which is a tributary of the main branch of Sixteen Mile Creek, traverses the entire length of the Gulfbeck Subdivision, from north to south. The watercourse is typically overgrown with vegetation and has only intermittent flows.

The geotechnical investigation completed for the subject site found that the topsoil depth ranged from 20 cm to 56 cm in thickness and the subsurface conditions consist of a stratum of silty clay till on top of shale bedrock of Queenston Formation. The bedrock surface generally slopes from the north to south and the top of the bedrock is at an elevation of approximately 165.0 m to 170.0 m in this area. The hydrogeological investigation of the subject site found that the soils had a hydralulic conductivity of 7.2 x10^{-7} cm/sec. This value is considered low and typical for clayey silt till deposits found in the area.



LEGEND

OVERLAND FLOW DIRECTION

STUDY AREA

TMIG
The Municipal Infrastructure Group Ltd
8800 Dufferin Street,
Suite 200
Vaughan, ON
L4K 0C5
p: 905.738.5700
f: 905.738.0065

BOYNE SURVEY MILTON

GULFBECK DEVELOPMENTS INC. SUBDVISION

EXISTING DRAINAGE

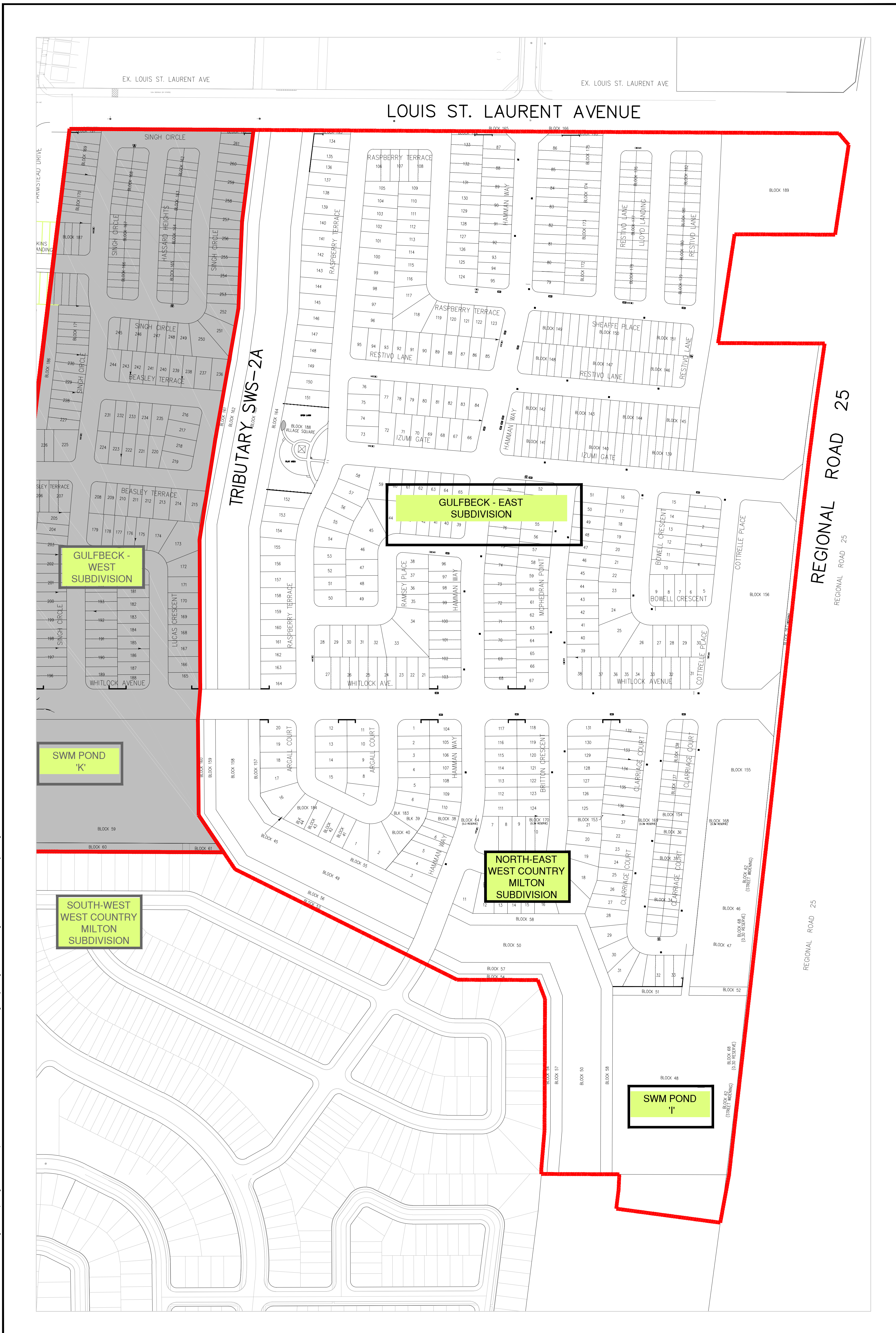
SCALE: N.T.S.		PROJECT No. 13138
DATE: SEPTEMBER 2016		
DESIGNED BY: K.T.	DRAWN BY:CAD	FIGURE No. 1-2
CHECKED BY: L.R.	CHECKED BY: L.R.	

1.4 Proposed Conditions

The proposed Gulfbeck Subdivision development plan for the lands tributary to SWM Pond 'I' includes a mix of residential dwellings comprised of single detached, semi-detached and townhouses, with right-of-ways (ROWs) varying from 16m to 26m. The proposed development plan also includes a park adjacent to the greenland channel block, as well as a major node block and residential/office blocks along the east boundary of the subject site. SWM Pond 'I' is located entirely within the West Country Milton Subdivision.

The proposed drainage area to SWM Pond 'I' is approximately 34.23 ha, consisting of a development area of 25.57 ha of the Gulfbeck Subdivision, 4.37 ha of the West Country Milton Subdivision (lands immediately north of SWM Pond 'I') and 2.53 ha of the Mattamy Framgard Subdivision (lands south of SWM Pond 'I'), as well as 1.76 ha from the future Regional Road 25 right-of-way. To ensure sufficient capacity in the SWM pond and the storm sewers the proposed Gulfbeck Subdivision drainage area of 25.57 ha includes the non-participating property located in the north-east portion of the Gulfbeck Subdivision and the external drainage area from Louis St. Laurent Avenue.

SWM Pond 'I' is proposed to discharge into tributary SWS-2-A. Tributary SWS-2-A is to be realigned in a channel block, which runs north-south through the Gulfbeck and West Country Milton Subdivision, then crosses the West Country Milton Subdivision before continuing to run north-south again. Natural channel design methods are proposed for the realigned channel and a report detailing the channel design will be submitted under separate cover. The proposed development plan is illustrated on **Figure 1-3.**



File : G:\Projects\2013\13138 - Great Gulf Gulfbeck Subdivision Phase 1\Drawings\SWM Report Figures\SWMP Sept 2016\FIGURE 1-3 - DEVELOPMENT PLAN POND I.dwg, Layout : figure 1-3 Date :Aug 31, 2016 - 10:41am, Edit By : rchung

TMIG
The Municipal Infrastructure Group Ltd
8800 Dufferin Street,
Suite 200
Vaughan, ON
L4K 0C5
p: 905.738.5700
f: 905.738.0065

BOYNE SURVEY MILTON

GULFBECK DEVELOPMENTS INC. SUBDVISION

DEVELOPMENT PLAN

SCALE: N.T.S.	PROJECT No. 13138
DATE: SEPTEMBER 2016	
DESIGNED BY: K.T.	DRAWN BY:CAD
CHECKED BY: L.R.	CHECKED BY: L.R.
FIGURE No. 1-3	

2 STORMWATER MANAGEMENT

The proposed stormwater management plan for the study area was set out in the Boyne Survey Block 2, Subwatershed Impact Study (SIS). Stormwater management (SWM) Pond 'I', located south of the Gulfbeck Subdivision on the West Country Milton lands, is to be designed to accommodate the major and minor storm system flows for an area of 34.23 ha. The proposed drainage area consists of a development area of 25.57 ha of the Gulfbeck Subdivision, 4.37 ha of the West Country Milton Subdivision (lands immediately north of SWM Pond 'I') and 2.53 ha of the Mattamy Framgard Subdivision (lands south of SWM Pond 'I'), as well as 1.76 ha from the future Regional Road 25 right-of-way.

The proposed SWM Pond 'I' is to be constructed concurrently with the subject site and will provide water quality treatment, erosion control and water quantity attenuation in accordance with the criteria set out in the Boyne Survey Block 2 SIS.

2.1 Design Criteria

The design criteria used for the stormwater management system for the study area was taken from the Town of Milton design standards, the MOE Stormwater Management Planning and Design Manual (SWMP&DM) and the Boyne Survey Block 2 SIS. These standards include:

- Minor system/storm sewers designed to convey 5 year storm flows;
- Storm events greater than the 5-year event up to the 100-year event will generally be conveyed overland to the pond via the roads;
- SWM Pond 'I' to provide Enhanced level water quality treatment, based on the MOE SWMP&DM;
- SWM Pond 'I' to provide erosion control and water quantity attenuation, based on the unit rates provided in the Boyne Survey Block 2 SIS. The unit rates are summarized in **Table 2-1**;

Table 2-1: Summary of Unitary Storage and Discharge Criteria for SWM Pond 'I'

Storage Component	Cumulative Storage Required (m^3/impervious ha)	Discharge (m^3/s/ha)
Erosion Control / Extended Detention	400	0.0006
25 Year	600	0.020
100 Year	825	0.050
Regional	1450	0.070

- SWM pond side slopes include: 5:1 slopes at the normal water level fringe (3m horizontally away from NWL); 3:1 slopes below the NWL fringe; and 4:1 slopes above the NWL fringe up to the Regional Water Level, as per the Town of Milton SWM pond design criteria;
- An emergency outlet for the Regional Storm flow will be provided in the SWM pond, such that all lots adjacent to the SWM facility will not be submerged during the Regional Storm event;
- A 4m wide maintenance access route from a municipal road with a maximum slope of 10:1 and a maximum cross-fall of 2% will be provided in the SWM pond. The access road will be used to facilitate the access to the forebay and outlet structure for maintenance;
- A 0.3 m free board will be provided in the SWM pond above the Regional Water Level; and
- A 7.5m SWM pond buffer is required when the SWM pond is adjacent to residential lots.

2.2 Hydrology

Hydrologic modeling was not completed as part of the stormwater management design for this site as SWM Pond facilities are sized based on the unit rates provided in the Boyne Survey Block 2 SIS. These rates were determined using the HSP-F hydrologic model and continuous simulation modelling completed by AMEC. The unitary storage and discharge criteria for SWM Pond 'I' are summarized above in **Table 2-1**.

2.3 Minor System Flows

The minor system is designed to accommodate the 5-year storm event flows as per the Town of Milton design standards; design sheets are included in **Appendix B**. The subject site has been graded in a manner such that flows greater than the 5 year storm event will be conveyed overland to the stormwater management facility.

Site grading constraints do not support the design of a full depth municipal storm sewer system capable of accommodating gravity connection to basement foundation systems. Accordingly, a large portion of the sewer system will be constructed at minimum depth, deep enough for frost protection.

In order to ensure that basements are properly drained, homes within this development will be equipped with sump pumps. In general, these sump pumps will drain to the rear yard. In cases where the lot drains to the front and a sidewalk is present, the sump pump will drain to a storm sewer lateral.

2.4 Major System Flows/ Right-of-Way Capacity

Overland flows will be directed via the subdivision road right-of-ways (ROWs) to SWM Pond 'I'. One overland flow inlet is proposed for SWM Pond 'I', which is provided at the north-east corner of the SWM pond block. The major system flows to the SWM pond will not exceed the width of the road allowance, and in no case will the depth of flow exceed 30 cm at the gutter or 15 cm at the crown, in accordance with the Town of Milton criteria. For all classes of roads, the product of depth of water (m) at the gutter and the velocity of flow (m/s) shall not exceed 0.65 m^2/s.

An overland flow analysis has been undertaken for the Gulfbeck and West Country Milton Subdivisions based on the site grading and current proposed development plan. The majority of the runoff from the subject site will be directed towards the SWM pond via Clarriage Court, which has a road ROW of 16 m wide. Therefore the capacity of Clarriage Court was verified, as it represents the worst case scenario.

The rational method was used to calculate the expected major flows along Clarriage Court. The maximum anticipated overland flow rate for Clarriage Court was calculated to be 2.55 m^3/s, based on an overall drainage area of 27.96 ha (total drainage area which would combine at the south-east corner of Clarriage Court). The maximum capacity of the 16 m ROW at a 0.8% slope, assuming a maximum ponding depth of 0.15m above the crown of the road, was calculated using the Manning's equation. The ROW capacity was calculated to be 2.77 m^3/s, which is greater than the anticipated maximum overland flow rate of 2.55 m^3/s. For further details, refer to the calculations provided in **Appendix A**.

The maximum anticipated overland flow rate for Clarriage Court was also calculated for the north-west segment and north-east segment of the road, as the slope is 0.5% in these segments. The maximum anticipated overland flow rate for north-west segment of Clarriage Court was calculated to be 2.04 m^3/s (based on an overall drainage area of 22.41 ha) and the maximum anticipated overland flow rate for north-east segment of Clarriage Court was calculated to be 0.51 m^3/s (based on an overall drainage area of 5.55 ha). The maximum capacity of the 16 m ROW at a 0.5% slope, assuming a maximum ponding depth of 0.15m above the crown of the road, was calculated using the Manning's equation. The ROW capacity was calculated to be 2.19 m^3/s, which is greater than the anticipated maximum overland flow rates of 2.04 m^3/s and 0.51 m^3/s. For further details, refer to the calculations provided in **Appendix A**.

The following table summarizes the results of this analysis:

Table 2-2: Overland Flow Summary

Pond ID	Proposed Roadway	Tributary Area (ha)	Max Flow within ROW (m^3/s)**	ROW Width (m)	Road Capacity (m^3/s)
SWM Pond ‘I’	Clarriage Court	27.96	2.55	16	2.77
SWM Pond ‘I’	Clarriage Court (north-west segment)	22.41	2.04	16	2.19
SWM Pond ‘I’	Clarriage Court (north-east segment)	5.55	0.51	16	2.19

Note: **The maximum flow within the ROW was calculated using the Rational Method for cross sections throughout the drainage system; the displayed value is the major system peak flow minus the minor system peak flow for the section of road with greatest tributary drainage area. The value shown is for the critical section of road determined by sampling throughout the entire subdivision.

The analysis demonstrates that there is sufficient road capacity for the anticipated overland flows. Detailed calculations are provided in **Appendix A.**

2.5 Stormwater Management Facility Design

As per the recommendations in the FSEMS and the Boyne Survey Block 2 SIS, the study area is tributary to SWM Pond ‘I’. SWM Pond ‘I’ is located at the low point of the study area. Flows from SWM Pond ‘I’ will discharge into the proposed realigned channel SWS-2-A. SWM Pond ‘I’ has been designed as a wet pond facility.

All efforts were made in designing the SWM pond to ensure that the configuration of both the forebay and the wet cell provide the maximum use of the block in terms of providing maximum storage volume and maximum flow length to maximize sediment settling and runoff cooling. SWM Pond ‘I’ has flow lengths of approximately 195 m from both the north and south inlets, and widths that vary from 30 m to 65 m, therefore the minimum provided flow length ratio is 3:1. In addition, the wet cell has been designed with a 1.5 m deep permanent pool that deepens to 3 m at the outlet structure and the SWM Pond outlet has been designed as a bottom draw outlet to ensure the flows out of the pond to the receiving watercourse are drawn from the cooler and deeper depths of the permanent pool. Landscaping plans that form part of the submission drawing set, incorporate a riparian planting strategy to provide shading of the pond embankments; enhancing the reduction to temperatures of the runoff leaving the SWM pond. A wetland pool will also be provided within the proposed realigned channel SWS-2-A at the pond outlet, this along with shading from the plantings will help mitigate the water temperature.

The proposed grading for the site is designed to direct the majority of major and minor system flows to SWM Pond ‘I’ prior to entering tributary SWS-2-A. All attempts were made to direct major and minor system drainage to the proposed SWM facility. Runoff from roof leaders will be directed to grassed areas.

Based on the proposed drainage plan and the proposed plans of development **Table 2-3** summarizes the total drainage areas contributing into SWM Pond ‘I’ and the corresponding runoff coefficients.

Table 2-3: Drainage Areas to SWM Pond 'I'

Area Breakdown / Proposed Land Use	Drainage Area	Imperviousness
	(ha)	*(%)*
Gulfbeck Subdivision		
Residential Lots (Single)	11.79	50%
Residential Lots (Semi-detached)	3.92	57%
Residential Lots (Townhouse)	4.70	79%
Major Node	2.27	93%
Residential / Office	1.49	86%
Park	0.24	57%
Open Space / Buffer	0.05	7%
Non-Participating Property	0.81	93%
External Road (Louis St. Laurent Ave)	0.30	90%
West Country Milton Subdivision		
Residential Lots (Single)	1.21	50%
Residential Lots (Semi-detached)	0.11	57%
Residential Lots (Townhouse)	0.47	79%
Residential / Office	0.86	86%
Open Space / Buffer	0.02	7%
SWM Pond	1.70	50%
Mattamy Framgard Subdivision		
Residential / Office	2.05	86%
Open Space / Buffer	0.08	7%
SWM Pond	0.40	50%
Regional Road 25		
External Road (Regional Road 25)	1.76	90%
Total Drainage Area (ha)	**34.23**	
Weighted Imperviousness (%)	**66.5%**	

Note: Impervious rates for each land use are based on the runoff coefficients specified in the Town standards.

The land use breakdown for the Mattamy Framgard Subdivision is based on the detailed storm drainage plan provided to TMIG on March 30, 2016.

TMIG The Municipal Infrastructure Group Ltd

8800 Dufferin Street,
Suite 200
Vaughan, ON
L4K 0C5
p: 905.738.5700
f: 905.738.0065

BOYNE SURVEY MILTON

GULFBECK DEVELOPMENTS INC. SUBDVISION

PROPOSED DRAINAGE AREA

SCALE: N.T.S.		PROJECT No.	13138
DATE: SEPTEMBER 2016			
DESIGNED BY: K.T.	DRAWN BY: CAD	FIGURE No.	2-1
CHECKED BY: L.R.	CHECKED BY: L.R.		

2.5.1 Facility Sizing

The proposed SWM facility has been designed as an enhanced quality wet pond, servicing post-development flows from the study area and the external areas. The total drainage area serviced by SWM Pond 'I' is 34.23 ha and has an average imperviousness of 66.5%. SWM Pond 'I' will provide water quality treatment, erosion control and water quantity attenuation in accordance with the criteria set out in the Town of Milton design manual, the MOE Stormwater Management Planning and Design Manual (SWMP&DM) and the Boyne Survey Block 2 SIS. The following sections detail the specific criteria that apply to each requirement, and **Section 2.5.5** summarizes the required and the provided values.

2.5.2 Water Quality Treatment

Water quality treatment has been provided in accordance with the MOE SWM Planning & Design Manual. SWM Pond 'I' has been designed to an Enhanced level of protection, which is consistent with the SWM design criteria. With a total tributary area of 34.23 ha and an average imperviousness of 66.5% the SWM facility requires a permanent pool volume of 5,917 m^3.

The total permanent pool volume provided within SWM Pond 'I' is 9,768 m^3, which exceeds the volume required. Detailed calculations are provided in **Appendix A.**

2.5.3 Erosion Control / Extended Detention

The erosion control criteria established in the FSEMS and the Boyne Survey Block 2 SIS stipulates targets of 400 m^3/impervious-ha of storage volume and an outflow control of 0.0006 m^3/s/ha. Based on a total contributing drainage area of 34.23 ha and an average imperviousness of 66.5%, the total required erosion control storage volume is 9,102 m^3, with a controlled outflow of 20.5 L/s.

An extended detention storage volume of 9,734 m^3, which exceeds the required storage volume, has been provided within SWM Pond 'I' between the elevations of 183.85 m (normal water level) and 184.75 m. The erosion control release rate is 20 L/s. Detailed calculations are provided in **Appendix A.**

2.5.4 Water Quantity Attenuation

The water quantity attenuation criteria were defined in the FSEMS (November 2015) and the Boyne Survey Block 2 SIS (July 2015), based on hydrologic modeling completed using the HSP-F hydrologic model. The unitary storage and discharge criteria for SWM Pond 'I' were summarized above in **Table 2-1**.

The require storage volumes and target release rates for SWM Pond 'I' were calculated based on a total contributing drainage area of 34.23 ha and an average imperviousness of 66.5%. The findings are summarized below in **Table 2-4**.

Table 2-4: Summary of Required Storage Volumes and Target Release Rates for SWM Pond 'I'

Storage Component	Cumulative Storage Required (m^3)	Discharge (m^3/s)
25 Year	13,653	0.685
100 Year	18,772	1.712
Regional	32,994	2.396

The quantity control volumes provided within the wet pond are 14,196 m^3 for the 25 year event, 18,916 m^3 for the 100 year event, and 34,644 m^3 for the Regional Storm event which exceeds the required storage volumes required.

Detailed calculations are provided in **Appendix A.**

2.5.5 Storage-Discharge Relationship

The storage-discharge relationship for SWM Pond 'I' is provided in **Table 2-5**, with the required storage volumes compared to the provided storage volumes and the associated release rates.

Table 2-5: Storage - Discharge Rates

Design Event	Required		Provided	
	Storage (m^3)	Discharge (m^3/s)	Storage (m^3)	Discharge (m^3/s)
Extended Detention	9,102	0.0205	9,734	0.020
25 year	13,653	0.685	14,196	0.668
100 year	18,772	1.712	18,916	1.705
Regional Storm	32,994	2.396	34,644	2.289

As shown in **Table 2-5**, the provided volumes are greater than the required volumes and the designed discharges are equal to or less than the required discharge; therefore, all the requirements have been satisfied. Detailed calculations are provided in **Appendix A.**

2.5.6 Forebay Sizing

The sediment forebays have been designed as per the MOE SWMP&DM to pre-treat the incoming flows. The sediment forebay design calculations are provided in **Appendix A**.

The north forebay has been designed with a settling length of 85 m and a depth of 1.5 m, which will allow sufficient time for suspended solids to settle out of the stormwater runoff. The south forebay has been designed with a settling length of 57 m and a depth of 1.5 m, which will allow sufficient time for suspended solids to settle out of the stormwater runoff. As per the recommendations of the MOE manual the forebay provided in facility SWM Pond 'I' has been designed with a minimum length to width ratio of 2:1.

2.5.7 Facility Outflow Details / Outlet Sizing

Discharge from SWM Pond 'I' will be provided through a multi-stage outlet configuration. The outlet design will ensure that outflows to tributary SWS-2-A are controlled to the target release rates for erosion control; the 25 year and the 100 year return period events; and the Regional Storm event.

A bottom draw reversed sloped pipe, controlled by an orifice plate, is proposed to provide erosion control / extended detention. The submerged end of the pipe will be installed with a Hickenbottom (perforated) pipe surrounded with a gravel jacket and filter cloth to prevent blockage of the perforated pipe. The orifice plate will control outflow from the pond to the erosion control target release rate. A 100 mm diameter orifice set at an invert elevation 183.85m, which is the normal water level of the permanent pool storage volume of the facility. The Hickenbottom pipe was designed with sufficient perforations to ensure that the 100 mm diameter orifice plate would control the flows (refer to calculations in **Appendix A**).

In addition to the erosion control orifice attenuating the outflow from SWM Pond 'I' flows will be discharged through two (2) ditch inlet catchbasins set at different elevations and controlled with orifice plates.

As can be seen in the detailed design drawings, SWM Pond 'I' has one (1) ditch inlet catch basin, with a top elevation set at the extended detention level of 184.75 m. Flows into this catchbasin will be conveyed to the control manhole via 675 mm diameter storm sewers, and will be controlled by a 550 mm diameter orifice set at an invert elevation of 183.85 m. This outlet structure will control the 2 through 25 year design storms.

The second ditch inlet catch basin has been set at an elevation of 185.10 m, which is above the 25 year storage level. Flows into this catchbasin will be conveyed to the control manhole via 675 mm diameter storm sewers, and will be controlled by a 610 mm diameter orifice set at an invert elevation of 183.85 m. This outlet structure will control the 25 through 100 year design storms and the Regional storm event.

The combination of the three outlet structures will ensure that the target release rates from all storm events up to the Regional storm are achieved.

The characteristics of the orifices within the control outlet structure are summarized in **Table 2-6**.

Table 2-6: Characteristics of the Orifices within the Control Structure

Control	Inv. Elev. (m)	Diameter (mm)	Lip Elev. (m)	Description
Orifice 1	183.85	100	n/a	Extended Detention
Orifice 2	183.85	550	184.75	2 yr to 25 yr
Orifice 3	183.85	610	185.10	25 yr to Regional

Detailed calculations for the sizing of the outlet structure are included in **Appendix A**. Control Manhole details are provided in **Drawings SWM-I04** and **SWM-I05** provided in **Appendix D.**

Controlled flows from the outlet structure will be directed through the 1200 mm diameter storm outfall pipe and into a wetland pool prior to spilling into the receiving watercourse (SWS-2-A). The SWM Pond outfall pipe is sized as a 1200 mm diameter storm pipe at 0.5% slope with a maximum capacity of 2.87 m^3/s.

2.5.8 Emergency Outlet

SWM Pond 'I' has been designed with an emergency spillway sized as a trapezoidal weir with a bottom width of 15 m and depth of 0.3 m. The weir is set at an invert of 186.50 m, equal to the expected Regional Storm water level in the pond. The emergency spillway will discharge into tributary SWS-2-A and has a maximum capacity of 5.35 m^3/s.

2.6 Pond Operation

The north inlet pipe to the SWM pond is a 1500 mm diameter concrete pipe with a 0.5% slope and is sized to adequately convey the 5 year design storm from the north drainage areas (Gulfbeck Subdivision and West Country Milton Subdivision lands). The major flows from the subdivisions will enter the SWM Pond via the overland flow routes. The south inlet pipe to the SWM Pond is a 1050 mm diameter concrete pipe with a 0.35% slope and is sized to adequately convey the major and minor storm system flows from the south drainage area (Mattamy Framgard Subdivision) as well as 1.76ha of drainage area from Regional Road 25.

A stage-storage-discharge curve was developed to model the performance of each outlet control under the different storm events to demonstrate that the target outflows are obtained. **Table 2-7** demonstrates that this pond operates by controlling outflows from the pond to a flow below the target values. Details of the stage-storage-discharge relationship are provided in **Appendix A.**

The controlled flows will be conveyed safely into the receiving watercourse via the proposed storm outfall pipe.

Table 2-7: Summary of SWM Pond I Operating Characteristics

Pond Component	Stage (m)	Depth above Permanent Pool (m)	Provided Storage Volume (m^3)	Discharge (m^3/s)
Permanent Pool	183.85	---	9,768	---
Extended Detention	184.75	0.90	9,734	0.020
25 year	185.10	1.25	14,196	0.668
100 year	185.45	1.60	18,916	1.705
Regional Storm	186.50	2.65	34,644	2.289

2.7 Access Road

A 4.0 m wide access road has been provided to access all structures in order to facilitate routine inspection and maintenance activities. The access road is graded with a cross-slope of 2%.

2.8 Buffer Area

In accordance with the Town of Milton standards, a 7.5 m buffer has been provided within the SWM pond block along all residential lots. Buffer blocks will contain access road and community trails.

3 LOW IMPACT DEVELOPMENT TECHNIQUES

The FSEMS requires that surface water recharge to groundwater be maintained at pre-development conditions. In order to mitigate the decrease in infiltration under post development conditions the Gulfbeck and West Country Milton Subdivisions include the implementation of Low Impact Development (LID) measures. The following LID measures have been included in the Subdivision design:

Increased Topsoil Depth

Increasing the topsoil depth means providing extra storage for runoff and hence increases infiltration and evapotranspiration opportunities. By implementing this measure, the soil storage available can be increased. An increased topsoil depth of 0.45 m has been implemented in the Gulfbeck and West Country Milton Subdivisions.

Roof Water to Grassed Areas

Directing the rooftop runoff to grassed areas increases the potential for much of this water to infiltrate. This measure prevents stormwater from directly entering the storm sewer system or flowing across connected impervious surfaces, such as driveways, that drain directly to a storm sewer. Drainage from the rooftop area can be directed to the front and rear yards, and conveyed through grass swales. Wherever possible the rooftops runoff is directed to pervious areas within the Gulfbeck and West Country Milton Subdivisions.

Grassed Swales

Grassed swales are open channels designed to convey, treat and attenuate stormwater runoff. The vegetation on the surface of the swale slows the runoff water to allow sedimentation, filtration, evapotranspiration and infiltration. Grassed swales are proposed within rear yards throughout the Gulfbeck and West Country Milton Subdivisions.

3.1 Water Balance

A site-wide water balance was completed for both the Gulfbeck and West Country Milton Subdivisions by R.J. Burnside, details are provided in the *Gulfbeck Post-Development Water Balance Letter,* dated August 10, 2016 (see **Appendix C**). The total pre-development recharge, the total post development recharge across the property with no mitigation and the infiltration deficit were estimated for each of the Subdivisions. **Table 3-1** summarizes the infiltration rates and targets for the Gulfbeck Subdivision and the north-east portion of the West Country Milton Subdivision. The infiltration deficit would be met through the use of LID strategies.

Table 3-1: Infiltration Rates and Targets

Site	Pre-Development (m^3/year)	Post-development (with no mitigation) (m^3/year)	LID Infiltration Target (m^3/year)
Gulfbeck Developments	62,000	28,000	34,000
West Country Milton Properties Ltd.	11,000	7,500	3,500

As shown in **Table 3-1**, without the implementation of LID there would be a total infiltration deficit of 37,500 m^3/year in the post-development condition. The LID strategies proposed to mitigate the infiltration deficit are increased topsoil depth, diverting roof water to grassed areas, and grassed swales within the rear yards. These strategies were analyzed by R.J. Burnside to quantify the infiltration volume potential and Burnside concluded that with the implementation of the LID strategies, 67,500 m^3/year of runoff will infiltrate. Therefore, with the LID mitigation there is a potential for a decrease in the post development water regime of 9%. Given the conservation assumptions and margins of error in the calculation, this is considered to be a very good result that demonstrates the effectiveness of such LID measures (see **Appendix C** for details).

4 THERMAL MITIGATION

Tributary SWS-2-A has been classified as supporting seasonal warm water fish communities within the Boyne Survey Block 2 SIS area and immediately downstream of Britannia Road. Overall, the fish communities supported within the watercourse are considered tolerant of poor water quality and resilient to warmer water temperatures. Tributary SWS-2A flows into an occupied Silver Shiner reach of stream several kilometres downstream. In light of the current fish communities within the study area and the potential for its improvement post development, considerations of thermal impacts from stormwater need to be considered.

Under post development conditions increased surface water temperatures may result from runoff from paved surfaces and from stormwater management (SWM) facilities. In order to mitigate these thermal inputs to the receiving watercourse the detailed design of SWM Pond 'I' has incorporated measures to mitigate thermal impacts to the receiving watercourse.

The Boyne Survey Block 2 SIS outlined a number of recommended measures to be considered in the detailed design of the SWM ponds. These measures, intended to provide the conditions within the watercourses to support healthy warm water fish communities, are summarized below:

- Increase the pool depth to approximately 3.0m from the permanent pool elevation in the vicinity of the outlet pipe. This will provide a reservoir of cool water, which will be discharged from the pond during the first approximate 10mm of an event. The MNRF has found this approach has been successful in reducing water temperatures;
- Increasing canopy cover within the SWM facility (particularly along the west and south sides);
- Outlet structures incorporating bottom draws/reverse sloped pipes;
- Cooling trenches between pond outlet and watercourses; and,
- Enhancement of riparian vegetation along the drainage path between the SWM facility outlet and the receiving watercourse.

The above thermal mitigation measures have been analyzed and the most effective ones have been incorporated into the design of SWM Pond 'I' to ensure that there are no negative impacts on the fish communities and that discharge temperatures are within the known temperature range for Silver Shiner. A summary of the thermal mitigation measures that have been incorporated into the SWM Pond design is provided below:

Increased Pool Depth at Outlet

Within SWM Pond 'I' the wet cell has been designed with a 1.5 m deep permanent pool that deepens to 3 m at the outlet structure. In order to provide the equivalent volume associated with runoff from the 10mm rainfall event the SWM facility requires a deep pool volume of 2,275 m^3. The deep pool volume provided within SWM Pond 'I' is 2,333 m^3, which is greater than the volume of runoff from a 10 mm rainfall event. Detailed calculations are provided in **Appendix A**.

Canopy Cover

The landscape plans for SWM Pond 'I' incorporate a riparian planting strategy to provide shading of the pond embankments and outlet structure; enhancing the reduction to temperatures of the runoff leaving the SWM pond.

Outlet Structures

The SWM Pond outlet has been designed as a reverse graded pipe that draws from the deep pool to ensure the flows out of the SWM pond to the receiving watercourse are drawn from the cooler and deeper depths of the permanent pool.

Drainage Path

The drainage path through SWM Pond 'I' has been maximized to the extent possible with the introduction of berms. The berms will be landscaped to allow for increased shading throughout the SWM Pond. A wetland pool has also been provided within the proposed realigned channel SWS-2-A at the pond outlet, this along with shading from the plantings will help mitigate the water temperature

Low Impact Development

LID measures that promote infiltration to the groundwater have been incorporated throughout the Gulfbeck and West Country Milton Subdivisions. This will reduce runoff to the SWM pond and will maintain groundwater contributions to the watercourse where applicable. The following LID measures have been implemented:

- Increased topsoil depth;
- Roof water to grassed areas; and
- Grassed swales.

SWM Facility Design Components

In order to ensure SWM Pond 'I' meets the temperature targets set out by the MNRF it was designed in accordance with the Thermal Mitigation checklist. The following components are present in the design of SWM Pond 'I':

- The permanent pool was designed with a depth of 1.5 m that deepens to 3 m at the outlet structure;
- The 3 m deep permanent pool at the outlet structure was sized to provide a deep pool that would accommodate the storage volume associated with the runoff from the 10 mm rainfall event;
- The SWM Pond outlet has been designed as a bottom draw outlet to ensure the flows out of the pond to the receiving watercourse are drawn from the cooler and deeper depths of the permanent pool;
- The total permanent pool volume provided within SWM Pond 'I' is 9,768 m^3;
- SWM Pond 'I' has an extended detention storage volume of 9,734 m^3 and an erosion control release rate is 20 L/s (0.02 m^3/s);
- The erosion control discharge duration is 268 hours;
- The 25 mm storm event discharge duration is 212 hours; and
- The discharge duration for pond storage volumes greater than 1.5m deep is 271 hours.

5 OPERATION AND MAINTENANCE

5.1 Inspections

As recommended in the MOE SWMP&DM, inspections should be made after significant storms (>10 mm) during the first two years of operation to ensure that the facility is functioning as per the design. It is anticipated that four inspections will be required per year. After the initial period and after proper operation has been confirmed, an inspection schedule can be established based on the observed operation of the pond. As a minimum requirement, the pond should be inspected annually.

5.2 Regular Operation and Maintenance Activities

Grass Cutting

Grass cutting is not recommended for the pond. Allowing grass to grow enhances the water quality and provides other benefits.

Weed Control

If weed control is required in order to remove a specific species, the weeds should be removed by hand.

Plantings

A vegetative community is required in three different locations – upland / flood, shoreline, and aquatic fringes. Planting methods and any replanting should be carried out in accordance with the approved Landscape Design and the recommendations of the MOE SWMP&DM, or as modified by the operating authority.

Trash Removal

Trash and debris should be removed by hand, performed as required based on inspections.

Sediment Removal

To ensure long-term effectiveness, the sediment that accumulates in the SWM facility should be periodically removed. The required frequency of sediment removal is dependent on two (2) factors:

The first is that the efficiency of total suspended solid (TSS) removal within the sediment forebay should not decrease below 5% of the MOE target removal efficiency for the specified pond type. As sediment accumulates in the SWM facility the removal efficiency decreases due to loss in storage volume. SWM Pond 'I' has been designed to provide enhanced level of protection in terms of water quality. As a result, the required TSS removal efficiency for the SWM facility is 80% and clean-out of the facility should be completed when the removal efficiency drops to 75% which corresponds to 58 years. Detailed calculations are provided in **Appendix A** for reference.

The second requirement is that SWM pond forebays should be cleaned out once one half of the starting storage volume has been taken up by accumulated sediment. The forebay Sediment Removal Frequency is generally much shorter than the overall clean out frequency for SWM facilities. The forebays are designed to trap the majority of the large sediment and debris, and typically requires clean out on a more frequent basis than the entire SWM facility. Calculations were completed to estimate the time it would take for half of the forebay storage to be filled with sediments for each of the forebays. The calculations show that clean out would be required after 16 years for the north forebay and 30 years for the south forebay.

Therefore, SWM Pond 'I' should be cleaned out every **16 years**. Detailed calculations are provided in **Appendix A** for reference.

To maintain proper hydraulic operation of the SWM facility, clean out should be completed when the accumulated sediments occupy approximately half the volume of the permanent pool within the forebay. It should be noted that the decision to undertake a forebay clean out should be based on the yearly inspection results for both the forebay and main cell. If the majority of accumulated sediments are found to be within the forebay and the main cell, than an entire SWM facility clean out may be required

The following methodology is proposed for the sediment removal from SWM Pond 'I':

- **Dewatering the Pond for Sediment Removal:** Dewatering the SWM facility for maintenance purposes should occur on a dry day when the pond contains only the permanent pool volume of water (i.e. max. elevation 183.85m). Dewatering of the SWM facility can be accomplished by pumping water from the permanent pool directly to downstream of the outlet structures. A standard 6-inch pump will convey a minimum flow of 1000 m^3/day (i.e. 12 l/s). Given the permanent pool volume of 9,768m^3, use of several pumps concurrently is recommended to reduce the time required to empty the pond.
- **Equipment:** A rubber tire backhoe or a track machine with wide tracks for mud would be required due to the wet, soft soil conditions which may be encountered within the SWM facility. The work should be done in the summertime on a dry day when the pond contains only the permanent pool volume.
- **Sediment Disposal:** As per the MOE SWM Manual (2003), all sediments removed from the pond should be tested to determine alternatives for disposal including depositing the material on land; landfill disposal; and hazardous waste disposal as per Ontario Regulation 347. A sample of the sediments removed is to be taken to a laboratory familiar with MOE's disposal guidelines and tested accordingly.

Safety

The pond should be provided with appropriate signage, as per the Town of Milton standard E-26, that warns the public of the presence of deep water and slopes. Two warning signs are to be provided at SWM Pond 'I', one at each of the access road entrances. Refer to drawing **SWM-I01** in **Appendix D** for details.

Fencing will be provided along SWM pond boundary adjacent to residential lots.

Landscape drawings will be prepared with strategic plantings around the perimeter of the pond in order to discourage direct access to the facility.

All inlets, outlets, structures, and headwalls will be provided with the appropriate grates, covers, and safety features in order to prevent public entry or tampering.

6 CONCLUSIONS

In summary, the proposed stormwater management strategy ensures that the required water quality treatment, erosion control and water quantity attenuation are provided for the Gulfbeck Subdivision and West Country Milton Subdivision lands tributary to SWM Pond 'I', such that the requirements outlined within the Town of Milton standards, the MOE SWMP design guidelines, the FSEMS and the SIS report are met.

It is our opinion that the information and level of detail contained in this report is adequate to obtain the required approvals for the stormwater management component of the proposed development. We trust you will find the contents of this report satisfactory. Please contact the undersigned if you have any questions or concerns.

Sincerely,
The Municipal Infrastructure Group Ltd.

Lana Russell, P.Eng.
Water Resource Engineer

A-3 Excerpts from DSEL Reports (2018)

david schaeffer engineering ltd
SMART SUBDIVISIONS™

600 Alden Road, Suite 500
Markham, Ontario L3R 0E7
Tel. (905) 475-3080
Fax (905) 475-3081
www.DSEL.ca

FUNCTIONAL SERVICING AND

STORMWATER MANAGEMENT REPORT

FOR

FRAMGARD NORTH MAJOR NODE
MATTAMY (MILTON WEST) LIMITED

TOWN OF MILTON
REGION OF HALTON

PROJECT NO. 17-953

APRIL 2018 – REV 1

FUNCTIONAL SERVICING AND
STORMWATER MANAGEMENT REPORT
FOR
FRAMGARD NORTH MAJOR NODE
MATTAMY (MILTON WEST) LIMITED

TOWN OF MILTON
REGION OF HALTON

APRIL 2018 – REV. 1
DSEL FILE: 17-953

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FOR
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**TOWN OF MILTON
REGION OF HALTON**

**APRIL 2018 – REV. 1
DSEL FILE: 17-953**

1.0 INTRODUCTION

David Schaeffer Engineering Limited (DSEL) has been retained by Mattamy (Milton West) Limited to prepare a Functional Servicing Report (FSR) in support of their application for re-zoning, for the purpose of developing a mixed use commercial and residential block in the Framgard North Major Node Development (FNMN).

The subject property is ***2.09 hectares*** in size, including a 0.34 hectare holdout property mid-block fronting Regional Road 25, and is bounded by Etheridge Avenue to the south, Regional Road 25 (Ontario St.) to the east, Tributary SWS-2-A to the west, and Tributary SWS-2-A-1 to the north, as illustrated in ***Figure 1,*** found in ***Drawings/Figures*** section of this report.

It should be noted that this proposal is to rezone the Mattamy-owned lands which encompass 1.75 ha of the FNMN. The 0.34 ha holdout is excluded from the current proposal. However, the concept plan encompasses the entire 2.09 ha site for the purpose of demonstrating the functional servicing for the ultimate proposed development. The subject lands are located within the Town of Milton in the Boyne Survey Block 2 area and are located in Subwatershed Impact Study Area 4***.***

The site will be developed for commercial and residential purposes and will be comprised of three buildings with associated surface and underground parking. Two of the buildings will be 6-storey residential buildings and the third will be a 2-storey commercial building. A copy of the architectural site plan has been included in the ***Drawings/Figures*** section of this report.

Access to the subject property is available from Etheridge Avenue and Regional Road 25. A series of 6.7 m access lanes, associated parking and sidewalks are proposed in the site as illustrated on the concept plan***.***

The objective of this report is to support the application for re-zoning by providing sufficient detail to demonstrate that the proposed development is supported by existing and proposed municipal servicing infrastructure, and that the site design conforms to Town of Milton and Region of Halton design criteria; Ministry of the Environment and Climate Change design guidelines; the requirements of Conservation Halton; and general industry practice.

2.0 PREVIOUS STUDIES AND REPORTS

The following material has been reviewed in order to identify the constraints governing development for the subject site:

- **Boyne Survey Block 2 Final Subwatershed Impact Study**
  MTE, Revised August 25, 2016.
  (SIS)

- **Water and Wastewater Functional Servicing Report for the Framgard Development**
  DSEL, July 2015.
  (Framgard FSR)

- **Functional Servicing and Stormwater Management Report for the Framgard South Major Node**
  DSEL, March 2018.
  (South Node FSR)

- **Gulfbeck Developments Subdivision – Stormwater Management Design Report – SWM Pond I**
  The Municipal Infrastructure Group Ltd., September 2016
  (SWM Report)

- **Town of Milton Engineering and Parks Standards Manual**
  Town of Milton, August 2014.
  (Milton Engineering Standards)

- **Water and Wastewater Linear Design Manual, Version 3.01**
  Region of Halton, July 2017.
  (Halton Region Design Criteria)

- ***Stormwater Management Planning and Design Manual***
  Ministry of the Environment, March 2003.
  (SWM Manual)

- ***O. Reg. 332/12 Ontario Building Code***
  Ministry of Municipal Affairs and Housing, 2012.

- ***Design Guidelines for Drinking Water Systems***
  Ministry of the Environment, 2008.

- ***Design Guidelines for Sewage Works***
  Ministry of the Environment, 2008.

- ***Erosion & Sediment Control Guidelines for Urban Construction***
  Toronto and Region Conservation Authority, December 2006.

- ***2017 Development Charges Background Study for Water, Wastewater, Roads & General Services Development Charges***
  Region of Halton, December 2017.
  (2017 DC Charges)

3.0 SANITARY SERVICING

3.1 Sanitary Sewer Design Criteria

The sanitary flow for the site has been designed according to the following ***Halton Region Design Criteria*** unless otherwise stated:

Sewer Design Criteria

Average dry weather flow	➢ 275 litres per capita per day
Infiltration	➢ 0.286 litres per second per hectare
Peaking Factor	➢ Peak Flow Factor – Harmon Formula
Maximum Capacity Used	➢ Maximum 85% full flow capacity

Population Criteria

Townhouse / Apartment 6 Storeys or Less	➢ 135 persons / ha
Light Commercial Services	➢ 90 persons / ha

3.2 Existing Wastewater Services

Existing sanitary sewers are available to the site, as shown below in ***Table 3-1***:

Table 3-1: Summary of Existing Wastewater Infrastructure

Street	Size
Etheridge Ave	200 mm
Reg Road 25	1350 mm

The existing wastewater mains are illustrated in ***Drawing EX-1*** found in the ***Drawings/Figures*** section of this report.

3.3 Proposed Wastewater Servicing

As per the water and wastewater functional servicing report prepared by DSEL (***Framgard FSR***), the subject site was contemplated to drain south to the 200 mm sanitary sewer within Etheridge Avenue, which flows east to the existing 1350 mm sanitary trunk within Regional Road 25.

A capacity analysis was completed for this segment of pipe up to the restricting leg of the 200 mm sanitary sewer. The most restrictive leg of sewer within Etheridge Avenue, between manhole 77A and 78A, has ***15.6 L/s*** of capacity (refer to ***as-built design sheet*** for the Etheridge Avenue sanitary sewer in ***Appendix A)***. As per the ***South Node FSR***, there is an additional flow of ***1.8 L/s*** to the sewer which was not originally accounted for in design. This results in the existing sewer having an available capacity of ***13.8 L/s***. The sanitary servicing scheme for the development is illustrated in ***Drawing SSP-1,*** in the ***Drawings/Figures*** section of this report.

Table 3-2 summarizes the estimated peak wastewater flows for the subject property compared to the flows contemplated in the detailed design of the Framgard Subdivision. See ***Appendix A*** for detailed calculations.

Table 3-2: Wastewater Allowance

Design Parameter	Framgard Design (L/s)	Proposed (L/s)
Estimated Average Dry Weather Flow	0.9	0.9
Estimated Peak Dry Weather Flow	3.7	3.7
Estimated Peak Wet Weather Flow	4.3	4.3

The subject property was accounted for as part of the ***Framgard FSR*** and in the detailed design of the Framgard Subdivision; the population density considered in the detailed design is equal to the density for the proposed concept plan based on the ***Halton Region Design Criteria***, (refer to ***Appendix A*** for Sanitary Drainage Plan for the Mattamy Framgard Phase 1 showing the subject site).

At the time of the ***Framgard FSR,*** the entire site was contemplated as a residential development. It should be noted that one of the proposed buildings is now contemplated to be a commercial building.

As described in ***Table 3-2***, the flow of ***4.3 L/s*** is the same that was contemplated in the detailed design of the subdivision and can be accommodated in the existing Etheridge Avenue sanitary sewer.

Detailed sanitary sewer design sheets are included in ***Appendix A***. The sanitary drainage plan is presented in ***Drawing SAN-1*** found in the ***Drawings/Figures*** section of this report.

4.0 WATER SERVICING

4.1 Water Supply Servicing Design Criteria

The water supply was designed according to the ***Halton Region Design Criteria***. By taking into consideration watermain sizing, depth, crossings, valves, hydrants, and service connections it was determined that adequate pressures and fire flows can be achieved. Water design flows have been analyzed by Municipal Engineering Solutions (see ***Appendix B*** for the report) with the criteria listed below:

Water Design Criteria

Average Daily Demand	➢ 275 litres per capita per day
Maximum Daily Demand Peaking Factor	➢ 2.25
Maximum Hourly Demand Peaking Factor	
Residential	➢ 4.00
Commercial	➢ 2.25

Halton Region Design Criteria requires domestic flows to be maintained between 40 psi (275 kPa) and 100 psi (690 kPa) and fire flow conditions maintained above 20 psi (140 kPa). The Ontario Building Code requires individual pressure regulating valves if static pressures are above 80 psi (550 kPa).

4.2 Existing Water Services

Existing watermains are available in the vicinity of the site as shown in ***Table 4-1***.

Table 4-1: Summary of Existing Watermains

Street	Size
Etheridge Ave	300 mm
Reg Road 25	750 mm
Reg Road 25	300 mm

The existing watermains are illustrated in ***Drawing EX-1***.

4.3 Proposed Water Supply

The subject site will be serviced by a single connection to an existing Valve Chamber within Etheridge Avenue. The watermain servicing scheme for the development is illustrated in ***Drawing SSP-1*** in the ***Drawings/Figures*** section of this report.

Water pressures were found to range between 430 kPa to 451 kPa during Average Day, 416 kPa to 437 kPa during Peak Hour and 173 L/s to 438 L/s of fire flow is available at 20 psi during the 2016 Scenario. During the 2031 scenario, pressures were found to range between 604 kPa to 624 kPa during Average Day, 543 kPa to 565 kPa during Peak Hour and 239 L/s to 748 L/s of fire flow is available at 20 psi. The modeled results by Municipal Engineering Solutions are in conformance with the ***Halton Region Design Criteria*** and exceed fire flow requirements for the site.

5.0 STORM DRAINAGE

5.1 Existing Drainage Patterns

Stormwater runoff from the subject property generally drains by sheet flow to the existing stormwater Tributary SWS-2-A channel to the west, to the existing stormwater Tributary SWS-2-

A-1 to the north and to the existing roadside ditch within the Regional Road 25 right-of-way to the east. The existing lands are currently undeveloped.

Flows that influence the watershed in which the subject property is located are further reviewed by the principal authority. The subject property is located within the Sixteen Mile Creek watershed and is therefore, subject to review by Conservation Halton.

5.2 Existing Storm Services

Existing storm sewers in the vicinity of the site are shown in ***Table 5-1*** below and illustrated on ***Drawing SWM-1*** in the ***Drawings/Figures*** section of this report.

Table 5-1: Summary of Existing Storm Sewers

Street	Size
To SWM Pond I	825 mm
Reg Road 25	West Roadside Ditch (Typ. 10 m Top Width, Depth Varies 0.5 m to 2.0 m)
Reg Road 25	300 mm and 375 mm

5.3 Conveyance of System Flows

The proposed underground parking garage extents are illustrated in ***Drawings GP-1*** and ***SSP-1***. Minor system flow is to be captured and conveyed by the internal mechanical system to be designed by the mechanical engineer at the detailed design stage.

Runoff up to the 100-year event will be conveyed by a proposed 825 mm diameter outlet pipe discharging to existing MH500 and ultimately SWM Pond I. Existing MH500 and the storm outfall to SWM Pond 1 were installed through the design and construction of the Gulfbeck Subdivision. The storm outfall between existing MH500 and SWM Pond I outlets under Tributary SWS-2-A-1.

The paved portion of the site plan’s road network will also provide a continuous overland flow route.

The anticipated 100-year flow from the site is ***749 L/s*** and the proposed 825 mm diameter outlet pipe at a slope of 0.40% has a capacity of ***908 L/s.*** The proposed pipe has capacity for the 100-year flow from the site.

The major system is illustrated in ***Drawing GP-1*** found in ***Drawings/Figures***.

Please refer to ***Appendix C*** for storm sewer calculations.

6.0 STORMWATER MANAGEMENT

6.1 Design Criteria and Guidelines

As per the ***SWM Report*** and ***SIS,*** SWM Pond I on the adjacent property will provide quality and quantity controls for the subject site.

6.2 Proposed Stormwater Management System

As discussed in ***Section 5.3***, the 100-year flow is proposed to be captured on site via a proposed 825 mm diameter storm sewer. The sewer is proposed to discharge to existing storm MH500, and ultimately outlet to SWM Pond I.

Flows from the site will discharge to existing SWM Pond I which was designed by TMIG as part of the Gulfbeck Subdivision to provide a TSS removal efficiency of 80% (Enhanced level water quality treatment based on the SWM Manual). In addition to water quality treatment, SWM Pond I was designed to provide erosion control and water quantity attenuation. Further details of the design of SWM Pond I and the associated water treatment are found in the ***SWM Report***.

As indicated in the ***SWM Report*** and per the detailed design for the Framgard Subdivision, the allowance for the Mattamy Framgard Subdivision in SWM Pond I was based on a drainage area of 2.05 ha at runoff coefficient of 0.80. Calculations based on the current site plan indicate that the flows will be based on 2.09 ha at a runoff coefficient of 0.74. This results in a 7.5% decrease for the anticipated stormwater flows from the site to SWM Pond I and confirms that there is capacity for the proposed development.

6.3 Regional Road 25 Roadside Ditch Re-Grading

It is proposed to raise the subject property above existing grade to ensure that positive drainage is realized after the re-development and urbanization of Regional Road 25. This results in required re-grading of the roadside ditch adjacent to the site. To minimize the use of temporary retaining walls along the site boundary, 2.5:1 sloping has been proposed along with minor ditch re-alignments. With the re-grading along the property line the depth of the ditch is being increased. It is anticipated that the increase in depth will be sufficient to convey the existing flow to the ditch. Grading of the ditch and subject site shown on drawing ***GP-1*** found in the ***Drawings/Figures*** section of this report.

Flow to the roadside ditch was summarized in the ***SIS*** and re-stated in the ***Table 6-1,*** below.

Table 6-1: Regional Road 25 Roadside Ditch Flow

	100-Year Flow (m^3/s)	Regional Event Flow (m^3/s)
Drainage Area 408	1.122	0.372

Refer to extracted stormwater drainage plan from the ***SIS*** for delineation of drainage areas 408 described above. A culvert is proposed at the site entrance to Regional Road 25, sized to convey the flow from Drainage Area 408 per the above table (see sizing details in ***Appendix C)***.

7.0 EROSION AND SEDIMENT CONTROL

An erosion and sediment control strategy will be implemented during the construction of services, including the following:

- Siltation control fencing
- Stone mud mat at all construction entrances
- Regular inspection and monitoring of the erosion and sediment control devices
- Removal and disposal of the erosion and sediment control devices after the site has been stabilized

8.0 CONCLUSIONS

This Functional Servicing and Stormwater Management Report provides an overview of the servicing plan for the Framgard North Major Node, located within the Town of Milton. This report demonstrates the availability of water, wastewater and storm services for the proposed site in accordance with Municipal and Regional criteria, and general industry practice.

We trust you will find the contents of this report satisfactory.

Prepared by,
David Schaeffer Engineering Ltd

A. Temelini

Anthony Temelini, E.I.T.

Reviewed by,
David Schaeffer Engineering Ltd

LICENSED PROFESSIONAL ENGINEER
J. Ailey
J. AILEY
100079299
2018/04/25
PROVINCE OF ONTARIO

Jennifer Ailey, P.Eng.

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david schaeffer engineering ltd
SMART SUBDIVISIONS™

600 Alden Road, Suite 500
Markham, Ontario L3R 0E7
Tel. (905) 475-3080
Fax (905) 475-3081
www.DSEL.ca

FUNCTIONAL SERVICING AND

STORMWATER MANAGEMENT REPORT

FOR

FRAMGARD SOUTH MAJOR NODE
MATTAMY (MILTON WEST) LIMITED

TOWN OF MILTON
REGION OF HALTON

PROJECT NO. 17-954

MARCH 2018 – REV 1

FUNCTIONAL SERVICING AND
STORMWATER MANAGEMENT REPORT
FOR
FRAMGARD SOUTH MAJOR NODE
MATTAMY (MILTON WEST) LIMITED

TOWN OF MILTON
REGION OF HALTON

MARCH 2018 – REV. 1
DSEL FILE: 17-954

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STORMWATER MANAGEMENT REPORT
FOR
FRAMGARD SOUTH MAJOR NODE

TOWN OF MILTON
REGION OF HALTON

MARCH 2018 – REV. 1
DSEL FILE: 17-954

1.0 INTRODUCTION

David Schaeffer Engineering Limited (DSEL) has been retained by Mattamy (Milton West) Limited to prepare a Functional Servicing Report (FSR) in support of their application for re-zoning, for the purpose of developing a residential block in the Framgard South Major Node Development (FSMN).

The subject property is ***2.4 hectares*** in size and is bounded by Etheridge Avenue to the north, Regional Road 25 (Ontario St.) to the east, Tributary SWS-2-A to the west, and Britannia Road to the south, as illustrated in ***Figure 1,*** found in ***Drawings/Figures*** section of this report. The subject lands are located within the Town of Milton in the Boyne Survey Block 2 area and are located in Subwatershed Impact Study Area 4***.***

The site will be developed for residential purposes and will be comprised of three buildings, 6 storeys each with associated surface and underground parking. A copy of the architectural site plan has been included in the ***Drawings/Figures*** section of this report.

Access to the subject property is available from Etheridge Avenue and Regional Road 25. A series of 6.7m access lanes, associated parking and sidewalks are proposed in the site as illustrated on the architectural site plan***.***

The objective of this report is to support the application for re-zoning by providing sufficient detail to demonstrate that the proposed development is supported by existing and proposed municipal servicing infrastructure, and that the site design conforms to Town of Milton and Region of Halton design criteria; Ministry of the Environment design guidelines; the requirements of Conservation Halton; and general industry practice.

2.0 PREVIOUS STUDIES AND REPORTS

The following material has been reviewed in order to identify the constraints governing development for the subject site:

- **Boyne Survey Block 2 Final Subwatershed Impact Study**
  MTE, Revised August 25, 2016.
  (SIS)

- **Water and Wastewater Functional Servicing Report for the Framgard Development**
  DSEL, July 2015.
  (Framgard FSR)

- **Stormwater Management Report for the Mattamy Framgard Subdivision to SWM Pond J**
  J.F. Sabourin and Associates Inc., May 2016
  (SWM Report)

- **Town of Milton Engineering and Parks Standards Manual**
  Town of Milton, August 2014.
  (Milton Engineering Standards)

- **Water and Wastewater Linear Design Manual, Version 3.00**
  Region of Halton, July 2017.
  (Halton Region Design Criteria)

- ***Stormwater Management Planning and Design Manual***
  Ministry of the Environment, March 2003.
  (SWM Manual)

- ***O. Reg. 332/12 Ontario Building Code***
  Ministry of Municipal Affairs and Housing, 2012.

- ***Design Guidelines for Drinking Water Systems***
  Ministry of the Environment, 2008.

- ***Design Guidelines for Sewage Works***
  Ministry of the Environment, 2008.

- ***Erosion & Sediment Control Guidelines for Urban Construction***
  Toronto and Region Conservation Authority, December 2006.

- ***2017 Development Charges Background Study for Water, Wastewater, Roads & General Services Development Charges***
  Region of Halton, December 2017.
  (*2017 DC Charges)*

3.0 SANITARY SERVICING

3.1 Sanitary Sewer Design Criteria

The sanitary flow for the site has been designed according to the following ***Halton Region Design Criteria*** unless otherwise stated:

Sewer Design Criteria

Average dry weather flow	➢ 275 litres per capita per day
Infiltration	➢ 0.286 litres per second per hectare
Peaking Factor	➢ Peak Flow Factor – Harmon Formula
Maximum Capacity Used	➢ Maximum 85% full flow capacity

Population Criteria

Townhouse / Apartment 6 Storeys or Less	➢ 135 persons / ha

3.2 Existing Wastewater Services

Existing sanitary sewers are available to the site, as shown below in ***Table 3-1***:

Table 3-1: Summary of Existing Wastewater Infrastructure

Street	Size
Etheridge Ave	200mm
Reg Road 25	1350mm
Britannia Rd	1200mm
Britannia Rd	675mm

The existing wastewater mains are illustrated in ***Drawing EX-1*** found in the ***Drawings/Figures*** section of this report.

3.3 Proposed Wastewater Servicing

As per the water and wastewater functional servicing report prepared by DSEL (***Framgard FSR***), the subject site was contemplated to drain south to the 1200mm sanitary trunk sewer within Britannia Road. The sanitary sewers servicing from Building B & C of the development will outlet to the existing sanitary 1200mm sanitary trunk sewer within Britannia Road. While Building A will outlet to the existing 200mm sanitary sewer within Etheridge Avenue, which flows east to the existing 1350mm sanitary trunk within Regional Road 25.

A capacity analysis was completed for this segment of pipe up to the connection to the 1350mm sanitary trunk since the development was not originally contemplated within the Etheridge Avenue sewer. The most restrictive leg of sewer within Etheridge Avenue, between manhole 77A and 78A, has ***16 L/s*** of capacity(refer to ***as-built design sheet*** for the Etheridge Avenue sanitary sewer in ***Appendix A)***. The sanitary servicing scheme for the development is illustrated in ***Drawing SSP-1,*** in the ***Drawings/Figures*** section of this report.

Table 3-2 summarizes the estimated average and peak wastewater flows for the subject property. See ***Appendix A*** for detailed calculations.

Table 3-2: Wastewater Allowance

Design Parameter	Total Flow to Etheridge Ave (L/s)	Total Flow to Britannia Rd (L/s)
Estimated Average Dry Weather Flow	0.4	0.7
Estimated Peak Dry Weather Flow	1.6	2.7
Estimated Peak Wet Weather Flow	1.8	3.2

The subject property was accounted for as part of the ***Framgard FSR*** and in the detailed design of the Framgard Subdivision; the population density considered in the detailed design is equal to the density for the proposed concept plan based on the ***Halton Region Design Criteria***, (refer to ***Appendix A*** for Sanitary Drainage Plan for the Mattamy Framgard Phase 1 showing the subject site).

As described in ***Table 3-2***, an additional flow to Etheridge Avenue of ***1.8 L/s*** can be accommodated in the existing Etheridge Avenue sanitary sewer.

Detailed sanitary sewer design sheets are included in ***Appendix A***. The sanitary drainage plan is presented in ***Drawing SAN-1*** found in the ***Drawings/Figures*** section of this report.

4.0 WATER SERVICING

4.1 Water Supply Servicing Design Criteria

The water supply was designed according to the ***Halton Region Design Criteria***. By taking into consideration watermain sizing, depth, crossings, valves, hydrants, and service connections it was determined that adequate pressures and fire flows can be achieved. Water design flows have been analyzed by Municipal Engineering Solutions (see ***Appendix B*** for the report) with the criteria listed below:

Water Design Criteria

Average Daily Demand	➢ 275 litres per capita per day
Maximum Daily Demand Peaking Factor	➢ 2.25
Maximum Hourly Demand Peaking Factor	
Residential	➢ 4.00

Halton Region Design Criteria requires domestic flows to be maintained between 40 psi (275 kPa) and 100 psi (690 kPa) and fire flow conditions maintained above 20 psi (140 kPa). The

Ontario Building Code requires individual pressure regulating valves if static pressures are above 80 psi (550 kPa).

4.2 Existing Water Services

Existing watermains are available in the vicinity of the site as shown in ***Table 4-1***.

Table 4-1: Summary of Existing Watermains

Street	Size
Etheridge Ave	300 mm
Reg Road 25	750 mm
Reg Road 25	300mm
Britannia Rd	750mm

The existing watermains are illustrated in ***Drawing EX-1***.

4.3 Proposed Water Supply

The subject site will be serviced by a single connection to an existing Valve Chamber within Etheridge Avenue. The watermain servicing scheme for the development is illustrated in ***Drawing SSP-1*** in the ***Drawings/Figures*** section of this report.

Water pressures were found to be 451 kPa and 436 kPa during Average Day and Peak Hour and 512 L/s of fire flow is available at 20 psi during the 2016 Scenario. During the 2031 scenario, pressures of 625 kPa and 564 kPa were found during Average Day and Peak hour and 934 L/s of fire flow is available at 20 psi. The modeled results by Municipal Engineering Solutions are in conformance with the ***Halton Region Design Criteria*** and exceed fire flow requirements for the site.

5.0 STORM DRAINAGE

5.1 Existing Drainage Patterns

Stormwater runoff from the subject property generally drains by sheet flow to the existing stormwater Tributary SWS-2-A channel to the west and to the existing roadside ditch within the Regional Road 25 right-of-way. The existing lands are currently undeveloped.

Flows that influence the watershed in which the subject property is located are further reviewed by the principal authority. The subject property is located within the Sixteen Mile Creek watershed and is therefore, subject to review by Conservation Halton.

5.2 Existing Storm Services

Existing storm sewers in the vicinity of the site are shown in ***Table 5-1*** below and illustrated on ***Drawing SWM-1*** in the ***Drawings/Figures*** section of this report.

Table 5-1: Summary of Existing Storm Sewers

Street	Size
Britannia Rd	300mm
Reg Road 25	West Roadside Ditch (Typ. 10m Top Width, Depth Varies 0.5m to 2.0m)
Reg Road 25	450mmw

5.3 Minor System Design

As per the ***SWM Report*** prepared by J.F. Sabourin and Associates Inc., flow from the future development south of Etheridge Avenue and east of Tributary SWS-2-A will require on-site controls prior to discharging to Tributary SWS-2-A. The proposed parking garage layout extends to the site limits and therefore, minor system flow is to be captured and conveyed by the internal mechanical system to be designed by the mechanical engineer at the detailed design stage.

5.4 Conveyance of Major System Flows

Major system runoff in excess of the minor system and up to the 100-year event will be conveyed through the paved portion of the site plan's road network via a continuous overland flow route. It is proposed to provide 100-year capture close to the outlet of the mechanical system to Tributary SWS-2-A to ensure controls for the 100-year and regional event can be provided by the internal cistern.

The major system is illustrated in ***Drawing GP-1*** found in ***Drawings/Figures***.

6.0 STORMWATER MANAGEMENT

6.1 Design Criteria and Guidelines

As per the ***SWM Pond J*** report, ***SIS*** and pre-consultation with the Town, Region, Conservation Authority, quantity controls are required in accordance with the ***SIS.*** The requirements are summarized in ***Table 6-1*** found below:

Table 6-1: Allowable Flow Rate & Control Storage per SIS

Design Storm	Unitary Controlled Flow (m^3/s/ha)	Unitary Flood Control Storage (m^3/imp.ha)	Allowable Release Rate (L/s)[1]	Flood Storage per SIS (m^3)[1 2]
Extended Detention (erosion control)	0.0006	400	1.4	826
25-Year	0.020	600	4.8	1238
100-Year	0.050	825	120	1702
Regional	0.070	1450	168	2992

1) Based on a 2.4 Ha Drainage Area
2) Based on runoff coefficient of 0.80, equal to a percent imperviousness of 86% calculated assuming C = 0.7 x %IMP + 0.2

The subject site is required to provide "enhanced" quality controls or 80% removal of Total Suspended Solids (TSS) per the ***SWM Manual.***

6.2 Model Assumptions

The following assumptions were made in the development of the post-development SWMHYMO model:

- Employed CALIB STANDHYD command and measured length of pervious flow path from the hydraulically most remote point to a pervious area. Impervious length calculated with the following equation: LGI = (Area / CLI)^0.5 where CLI = 1.5
- Pervious and Impervious slopes determined from Grading Plan for each catchment
- Initial abstraction (Impervious IA = 0.80mm; Pervious IA = 1.50mm)
- Horton's infiltration for soil loss (Fo = 76.2 mm/hr; Fc = 13.2 mm/hr, DCAY = 4.14/hr)
- Estimated % impervious area based on actual imperviousness of development
- Manning's coefficient (Pervious = 0.25; Impervious 0.013)
- Stage-Storage curve assumes pumped release rate from Cistern, at the max ponding of each storm event (25mm, 25-year, 100-year, regional) pumped release rate increases.
- 4hr Chicago storm distribution determined to be critical storm event

6.3 Proposed Stormwater Management System

As discussed in ***Section 5.3*** & ***Section 5.4***, flow is proposed to be conveyed by the building mechanical system and overland to 100-year capture locations where quantity controls are provided by an internal cistern. The cistern is proposed to be pumped from the parking garage level and discharge to a storm sewer, directed to the Tributary SWS-2-A.

Prior to discharge into the cistern, flow will be controlled through rooftop storage and by surface storage within sags above the parking garage. Roof controls and surface storage to be confirmed during detailed design with detailed grading and building mechanical design.

The internal cistern will be designed such that the pumped rate will equal the allowable release rates described in ***Table 6-1***. Refer to ***Table 6-2*** below for the proposed inflow, required storage and pumped flow rate from cistern.

Table 6-2: Proposed Cistern Inflow, Storage and Outflow

Design Storm	Inflow (m^3/s)	Required Storage (m^3)	Outflow (L/s)
Extended Detention (erosion control)	0.274	489	1.4
25-Year 4hr Chicago Distribution	0.869	1056	4.8
100-Year 4hr Chicago Distribution	1.071	1198	120
Regional (Hurricane Hazel)	0.341	1954	168

As shown above, the total storage required to attenuate the flow to the allowable release rate is ***1954m³*** in the Regional Storm event (refer to ***Appendix C*** for model input and output files). The storage will be provided through rooftop storage, surface ponding and cistern storage. The interaction of the 3 types of storage will be modeled during detailed design and may result in changes to the total required storage.

Quality controls are proposed through the use of a Jellyfish Filter System (or approved equivalent) downstream of the proposed cistern to provide a minimum of 80% TSS Removal. Please refer to ***Appendix C*** for sizing report prepared by the manufacturer for the proposed quality control unit.

6.4 Regional Road 25 Roadside Ditch Re-Grading

It is proposed to raise the subject property above existing grade to ensure that positive drainage is realized after the re-development and urbanization of Regional Road 25. This results in required re-grading of the roadside ditch adjacent to the site. To minimize the use of temporary retaining walls along the site boundary, a 2.5:1 sloping has been proposed and along with minor ditch re-alignments. A temporary retaining wall will be required at the southerly edge of the site, fronting the re-aligned ditch, to accommodate the proposed grade of the subject property. With the re-grading along the property line the depth of the ditch is being increased. It is anticipated that the increase in depth will be sufficient to convey the existing flow to the ditch. Grading of the ditch and subject site shown on drawing ***GP-1*** found in the ***Drawings/Figures*** section of this report.

Flow to the roadside ditch was summarized in the ***SIS*** and re-stated in the ***Table 6-3,*** below.

Table 6-3: Regional Road 25 Roadside Ditch Flow

	100-Year Flow (m^3/s)	Regional Event Flow (m^3/s)
Drainage Area 407	1.713	0.569
Drainage Area 408	1.122	0.372
Combined Drainage Area	2.798	0.927

Refer to extracted stormwater drainage plan from the ***SIS*** for delineation of drainage areas 407 and 408 described above. A culvert is proposed at the site entrance to Regional Road 25, sized to convey the flow from Drainage Area 408 per the above table (see sizing details in ***Appendix C***).

7.0 EROSION AND SEDIMENT CONTROL

An erosion and sediment control strategy will be implemented during the construction of services, including the following:

- Siltation control fencing
- Stone mud mat at all construction entrances
- Regular inspection and monitoring of the erosion and sediment control devices
- Removal and disposal of the erosion and sediment control devices after the site has been stabilized

8.0 CONCLUSIONS

This Functional Servicing and Stormwater Management Report provides an overview of the servicing plan for the Framgard South Major Node, located within the Town of Milton. This report demonstrates the availability of water, wastewater and storm services for the proposed site in accordance with Municipal and Regional criteria, and general industry practice.

We trust you will find the contents of this report satisfactory.

Prepared by, **David Schaeffer Engineering Ltd**	Reviewed by, **David Schaeffer Engineering Ltd**
LICENSED PROFESSIONAL ENGINEER S. L. MERRICK 100188523 2018-03-23 PROVINCE OF ONTARIO JOB #17-954	2018-03-23 #17-954
Steven L. Merrick, P.Eng.	Adam D. Fobert, P.Eng.

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APPENDIX

B

SWM Calculations

	Stormwater Management Calculations	Project:	Framgard North and South Block	No.:	231-00962
	Post-Development Area Take Off - By Catchment Areas	By: AM		Date: 2024-01-19	Page: 1
		Checked: GW			

South Block (Catchment 201)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,922.9	0.90	100%	28%
Green Roof	-	0.25	0%	0%
Soft Landscaping	561.6	0.25	0%	8%
Soft Landscaping (Above Parking Structure)	236.3	0.25	0%	3%
At-Grade Impervious	4,174.8	0.90	100%	61%
Total Area:	**6,895.6**	**0.82**	**88.4%**	**100%**

South Block (Catchment 202)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,123.6	0.90	100%	22%
Green Roof	-	0.25	0%	0%
Soft Landscaping	381.9	0.25	0%	8%
Soft Landscaping (Above Parking Structure)	106.0	0.25	0%	2%
At-Grade Impervious	3,387.8	0.90	100%	68%
Total Area:	**4,999.3**	**0.84**	**90.2%**	**100%**

South Block (Catchment 203)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,123.6	0.90	100%	22%
Green Roof	-	0.25	0%	0%
Soft Landscaping	930.0	0.25	0%	19%
Soft Landscaping (Above Parking Structure)	60.5	0.25	0%	1%
At-Grade Impervious	2,889.7	0.90	100%	58%
Total Area:	**5,003.9**	**0.77**	**80.2%**	**100%**

South Block (Catchment 204)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,216.7	0.90	100%	17%
Green Roof	-	0.25	0%	0%
Soft Landscaping	1,464.2	0.25	0%	21%
Soft Landscaping (Above Parking Structure)	80.3	0.25	0%	1%
At-Grade Impervious	4,343.1	0.90	100%	61%
Total Area:	**7,104.4**	**0.76**	**78.3%**	**100%**

North Block (Catchment 205)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,776.4	0.90	100%	23%
Green Roof	-	0.25	0%	0%
Soft Landscaping	966.3	0.25	0%	13%
Soft Landscaping (Above Parking Structure)	190.4	0.25	0%	2%
At-Grade Impervious	4,750.3	0.90	100%	62%
Total Area:	**7,683.4**	**0.80**	**84.9%**	**100%**

North Block (Catchment 206)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,123.6	0.90	100%	27%
Green Roof	-	0.25	0%	0%
Soft Landscaping	724.9	0.25	0%	17%
Soft Landscaping (Above Parking Structure)	180.7	0.25	0%	4%
At-Grade Impervious	2,201.7	0.90	100%	52%
Total Area:	**4,231.0**	**0.76**	**78.6%**	**100%**

North Block (Catchment 207)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	1,123.6	0.90	100%	20%
Green Roof	-	0.25	0%	0%
Soft Landscaping	834.2	0.25	0%	15%
Soft Landscaping (Above Parking Structure)	305.7	0.25	0%	5%
At-Grade Impervious	3,329.9	0.90	100%	60%
Total Area:	**5,593.5**	**0.77**	**79.6%**	**100%**

North Block (Catchment 208 (Holdout))

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	-	0.90	100%	0%
Green Roof	-	0.25	0%	0%
Soft Landscaping*	255.0	0.25	0%	7%
Soft Landscaping (Above Parking Structure)	-	0.25	0%	0%
At-Grade Impervious	3,392.8	0.90	100%	93%
Total Area:	**3,647.8**	**0.85**	**93.0%**	**100%**

*Assumed 255m2 of landscaped area and a design runoff coefficient of 0.85 for Catchment 208

	Stormwater Management Calculations	Project: Framgard North and South Block	No.: 231-00962
	Post-Development Area Take Off - By SWM Facility	By: AM	Date: 2024-01-19
		Checked: GW	Page: 2

South Block

Cistern A (201,202)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	3,047	0.90	100%	26%
Green Roof	-	0.25	0%	0%
Soft Landscaping	943	0.25	0%	8%
Soft Landscaping (Above Parking Structure)	342	0.25	0%	3%
At-Grade Impervious	7,563	0.90	100%	64%
Total Area:	**11,895**	**0.83**	**89%**	**100%**

Cistern B (203,204)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	2,340	0.90	100%	19%
Green Roof	-	0.25	0%	0%
Soft Landscaping	2,394	0.25	0%	20%
Soft Landscaping (Above Parking Structure)	141	0.25	0%	1%
At-Grade Impervious	7,233	0.90	100%	60%
Total Area:	**12,108**	**0.76**	**79%**	**100%**

North Block

SWM Pond "I" (205,206,207,208)

Land Use	Area (m2)	Runoff C	% Impervious	% Coverage
Impervious Roof Area	4,024	0.90	100%	19%
Green Roof	-	0.25	0%	0%
Soft Landscaping	2,780	0.25	0%	13%
Soft Landscaping (Above Parking Structure)	677			3%
At-Grade Impervious	13,675	0.90	100%	65%
Total Area:	**21,156**	**0.79**	**84%**	**100%**

Calculation of existing runoff rate is undertaken using the Rational Method: Q = 2.78 CIA

Where: Q = Peak flow rate (litres/second)
C = Runoff coefficient
I = Rainfall intensity (mm/hour)
A = Catchment area (hectares)

Project Area, A **2.40** hectares
Runoff Coef, C* 0.25

$$I = \frac{A}{(t + B)^c}$$

Where: A, B and C = Parameters defined in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual
I = Rainfall intensity (mm/hour)
t = Time of concentration (minutes)

Return Period (Years)	2	5	10	25	50	100
A	779.0	959.0	1089.0	1234.0	1323.0	1435.0
B	6.0	5.7	5.7	5.5	5.3	5.2
C	0.82	0.80	0.80	0.79	0.78	0.78
T (mins) *	10	10	10	10	10	10
I (mm/hr)	80.1	105.3	121.8	143.0	158.2	174.1
Q (litres/sec)	133.6	175.6	203.2	238.6	263.9	290.4
Q (m3/sec)	0.134	0.176	0.203	0.239	0.264	0.290

* Note recommended value for time of concentration is 10 minutes
as stated in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual

Calculation of existing runoff rate is undertaken using the Allowable Flow Rate as per SIS: *Q = UA*

Where: Q = Peak flow rate (litres/second)

U = Unitary Controlled Flow (meters3/second/hectares)

A = Catchment area (hectares)

Project Area, A **2.40** hectares

*Note the following table is taken from the Town's SIS Report

Design Storm	U (m3/sec/ha)
Erosion Control / Extended Detention	0.0006
25-Year	0.02
100-Year	0.05
Regional	0.07

Design Storm	Q (m3/sec)	Q (litres/sec)
Erosion Control / Extended Detention	0.001	1.4
25-Year	0.048	48.0
100-Year	0.120	120.0
Regional	0.168	168.0

Calculation of existing runoff rate is undertaken using the Rational Method: Q = 2.78 CIA

Where: Q = Peak flow rate (litres/second)
C = Runoff coefficient
I = Rainfall intensity (mm/hour)
A = Catchment area (hectares)

Project Area, A **1.75** hectares
Runoff Coef, C* 0.25

$$I = \frac{A}{(t+B)^c}$$

Where: A, B and C = Parameters defined in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual
I = Rainfall intensity (mm/hour)
t = Time of concentration (minutes)

Return Period (Years)	2	5	10	25	50	100
A	779.0	959.0	1089.0	1234.0	1323.0	1435.0
B	6.0	5.7	5.7	5.5	5.3	5.2
C	0.82	0.80	0.80	0.79	0.78	0.78
T (mins) *	10	10	10	10	10	10
I (mm/hr)	80.1	105.3	121.8	143.0	158.2	174.1
Q (litres/sec)	97.4	128.1	148.2	174.0	192.5	211.8
Q (m3/sec)	0.097	0.128	0.148	0.174	0.192	0.212

* Note recommended value for time of concentration is 10 minutes
as stated in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual

Calculation of existing runoff rate is undertaken using the Rational Method: Q = 2.78 CIA

Where: Q = Peak flow rate (litres/second)

C = Runoff coefficient

I = Rainfall intensity (mm/hour)

A = Catchment area (hectares)

Project Area, A **0.36** hectares

Runoff Coef, C* 0.85

$$I = \frac{A}{(t + B)^c}$$

Where: A, B and C = Parameters defined in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual

I = Rainfall intensity (mm/hour)

t = Time of concentration (minutes)

Return Period (Years)	2	5	10	25	50	100
A	779.0	959.0	1089.0	1234.0	1323.0	1435.0
B	6.0	5.7	5.7	5.5	5.3	5.2
C	0.82	0.80	0.80	0.79	0.78	0.78
T (mins) *	10	10	10	10	10	10
I (mm/hr)	80.1	105.3	121.8	143.0	158.2	174.1
Q (litres/sec)	69.4	91.2	105.6	123.9	137.1	150.9
Q (m3/sec)	0.069	0.091	0.106	0.124	0.137	0.151

* Note recommended value for time of concentration is 10 minutes

as stated in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual

Total Existing Release Rate from the North Block (Catchment 103 and EXT1)

Return Period (Years)	2	5	10	25	50	100
Q (litres/sec)	**166.8**	**219.3**	**253.8**	**297.9**	**329.6**	**362.7**

Calculation of existing runoff rate is undertaken using the Allowable Flow Rate as per SIS: *Q = UA*

Where: Q = Peak flow rate (litres/second)

U = Unitary Controlled Flow Rates (cubic meters/second/hectares)

A = Catchment area (hectares)

Project Area, A **1.19** hectares

*Note the following table is taken from the Town's SIS Report

Design Storm	U (m3/sec/ha)	
Erosion Control / Extended Detention	0.0006	25mm Design Storm Chicago
25-Year	0.02	25yr 4hr 10min Chicago
100-Year	0.05	100yr 4hr 10min Chicago
Regional	0.07	Hazel

Design Storm	Q (m3/sec)	Q (litres/sec)
Erosion Control / Extended Detention	0.001	0.71
25-Year	0.024	23.79
100-Year	0.059	59.47
Regional	0.083	83.26

Notes:

- The Unitary Controlled Flow Rate were taken from Table 4.2 from the "Boyne Survey Block 2 - Subwatershed Impact Study" prepared by MTE Consultants Inc. dated Aug 25, 2016
- The 4hr Chicago Storm was selected and noted as per the previous reports prepared for the development by DSEL and the SIS Study prepared by MTE to size the culvert. 4hr Chicago storm was derived from the Town of Milton IDF Parameters

Underground Storage Required - Catchments 201 & 202 (Phases 1 and 2)

Lowest Elev. in the Creek (m)	182.25	m	(in front of Phase 1)
Lowest Elev. at the Property Line facing Cistern (m)	185.50	m	(in front of Phase 1)
Elev. of At-grade Parking on top of Cistern (m)	188.20	m	for Phase 1 (based on Arch Plan dated Dec 21, 2023)
Elev. of ground level east of Parking Structure (m)	186.00	m	for Phase 1 (based on Arch Plan dated Dec 21, 2023)
Elev. of P1 Level (m)	184.00	m	for Phase 1 (based on Arch Plan dated Dec 21, 2023)
Elev. of P2 Level (m)	181.05	m	for Phase 1 (based on Arch Plan dated Dec 21, 2023)
Assumed Thickness of Slab and Pavement on Top	1.00	m	
Max. Height of P1 Cistern (m)	3.20	m	
Max. Height of P2 Cistern (m)	6.15	m	
Area of Cistern (m^2)	**280**	**m^2**	
Height of Cistern (m)	**5.5**	**m**	
Total Volume (m^3)	**1540.0**	**m^3**	

TABLE 4.2 - GENERAL DESIGN CRITERIA – FLOOD CONTROL CHARACTERISTICS

Characteristic		SWS-1-A	SWS-2-A
Unitary Flood Control Storage (m^3/Imp. ha)	Extended Detention (Erosion Control)	400	400
	Up to 25-Year Stage	650	600
	Up to 100-Year Stage	950	825
	Regional Storm	1825	1450
Unitary Controlled Flow Rate (m^3/s/ ha)	Extended Detention (Erosion Control)	0.0006	0.0006
	Up to 25-Year Stage	0.010	0.020
	Up to 100-Year Stage	0.035	0.050
	Regional Storm	0.052	0.070

**Data included in Table 4.2 is based on hydrologic verification for Block 2 completed by Amec (refer to June 15/15 correspondence included in Appendix C2). This data includes the impact of the Indian Creek diversion to SWS-1-A.*

Catchment Area: 1.19 ha

Design Storm	Allowable Release Rate (m3/s)	Estimated Storage (m3)
Erosion Control / Extended Detention	0.0007	476
25-Year	0.0238	714
100-Year	0.0595	981
Regional	0.0833	1725

Note: Storage-Discharge Curve inputed to VO Model

Rainfall Event	Target Release Rate	Flow (m^3/s)	Storage (m^3)
Erosion Control / Extended Detention	0.0007	0.0007	259
25-Year	0.0238	0.0140	624
100-Year	0.0595	0.0230	726
Regional	0.0833	0.0750	1442

Note: from VO Modelling

	Stormwater Management Calculations	Project:	Framgard North and South Block	No.:	231-00962	
	Allowable Offsite Discharge Rate to the SWS-2-A Channel (Catchments 203 and 204)	By:	AM	Date:	2024-01-19	Page:
		Checked:	GW			9

Calculation of existing runoff rate is undertaken using the Allowable Flow Rate as per SIS: *Q = UA*

Where: Q = Peak flow rate (litres/second)

U = Unitary Controlled Flow Rates (cubic meters/second/hectares)

A = Catchment area (hectares)

Project Area, A **1.21** hectares

*Note the following table is taken from the Town's SIS Report

Design Storm	U (m3/sec/ha)	
Erosion Control / Extended Detention	0.0006	25mm Design Storm Chicago
25-Year	0.02	25yr 4hr 10min Chicago
100-Year	0.05	100yr 4hr 10min Chicago
Regional	0.07	Hazel

Design Storm	Q (m3/sec)	Q (litres/sec)
Erosion Control / Extended Detention	0.001	0.73
25-Year	0.024	24.22
100-Year	0.061	60.54
Regional	0.085	84.76

Notes:

- The Unitary Controlled Flow Rate were taken from Table 4.2 from the "Boyne Survey Block 2 - Subwatershed Impact Study" prepared by MTE Consultants Inc. dated Aug 25, 2016
- The 4hr Chicago Storm was selected and noted as per the previous reports prepared for the development by DSEL and the SIS Study prepared by MTE to size the culvert. 4hr Chicago storm was derived from the Town of Milton IDF Parameters

Underground Storage Required - Catchments 203 & 204 (Phases 3 and 4)

Lowest Elev. in the Creek (m)	182.00	m	(in front of Phase 3)
Lowest Elev. at the Property Line facing cistern (m)	185.30	m	(in front of Phase 3)
Elev. of At-grade Parking on top of Cistern (m)	187.55	m	for Phases 3-4 (based on Arch Plan dated Dec 21, 2023)
Elev. of ground level east of Parking Structure (m)	185.65	m	for Phase 3 (based on Arch Plan dated Dec 21, 2023)
Elev. of P1 Level (m)	184.00	m	for Phases 3-4 (based on Arch Plan dated Dec 21, 2023)
Elev. of P2 Level (m)	181.05	m	for Phases 3-4 (based on Arch Plan dated Dec 21, 2023)
Assumed Thickness of Slab and Pavement on Top	1.00	m	
Max. Height of P1 Cistern (m)	2.55	m	
Max. Height of P2 Cistern (m)	5.50	m	
Area of Cistern (m^2)	**280**	m^2	
Height of Cistern (m)	**5.5**	**m**	
Total Volume (m^3)	**1540.0**	m^3	

TABLE 4.2 - GENERAL DESIGN CRITERIA – FLOOD CONTROL CHARACTERISTICS

Characteristic		SWS-1-A	SWS-2-A
Unitary Flood Control Storage (m^3/Imp. ha)	Extended Detention (Erosion Control)	400	400
	Up to 25-Year Stage	650	600
	Up to 100-Year Stage	950	825
	Regional Storm	1825	1450
Unitary Controlled Flow Rate (m^3/s/ ha)	Extended Detention (Erosion Control)	0.0006	0.0006
	Up to 25-Year Stage	0.010	0.020
	Up to 100-Year Stage	0.035	0.050
	Regional Storm	0.052	0.070

**Data included in Table 4.2 is based on hydrologic verification for Block 2 completed by Amec (refer to June 15/15 correspondence included in Appendix C2). This data includes the impact of the Indian Creek diversion to SWS-1-A.*

Catchment Area: 1.21 ha

Design Storm	Allowable Release Rate (m3/s)	Estimated Storage (m3)
Erosion Control / Extended Detention	0.0007	484
25-Year	0.0242	726
100-Year	0.0605	999
Regional	0.0848	1756

Note: Storage-Discharge Curve inputed to VO Model

Rainfall Event	Target Release Rate	Flow (m^3/s)	Storage (m^3)
Erosion Control / Extended Detention	0.0007	0.0007	245
25-Year	0.0242	0.0130	612
100-Year	0.0605	0.0230	712
Regional	0.0848	0.0730	1451

Note: from VO Modelling

Calculation of existing runoff rate is undertaken using the Rational Method: Q = 2.78 CIA

Where: Q = Peak flow rate (litres/second)
C = Runoff coefficient
I = Rainfall intensity (mm/hour)
A = Catchment area (hectares)

Project Area, A **2.13** hectares
Runoff Coef, C* 0.79

Note: Area and Runoff Coefficient Based on TMIG SWM Report for SWM Pond "I"

$$I = \frac{A}{(t+B)^c}$$

Where: A, B and C = Parameters defined in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual
I = Rainfall intensity (mm/hour)
t = Time of concentration (minutes)

Return Period (Years)	2	5	10	25	50	100
A	779.0	959.0	1089.0	1234.0	1323.0	1435.0
B	6.0	5.7	5.7	5.5	5.3	5.2
C	0.82	0.80	0.80	0.79	0.78	0.78
T (mins) *	10	10	10	10	10	10
I (mm/hr)	80.1	105.3	121.8	143.0	158.2	174.1
Q (litres/sec)	374.5	492.4	569.8	669.0	740.0	814.4
Q (m3/sec)	0.375	0.492	0.570	0.669	0.740	0.814

* Note recommended value for time of concentration is 10 minutes
as stated in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual

	Stormwater Management Calculations	Project:	Framgard North and South Block	No.:	231-00962	
	Post-Development Offsite Discharge Rate to SWM Facility I (North Block - Catchments 205-	By:	AM	Date:	2024-01-19	Page: 12
		Checked:	GW			

Calculation of existing runoff rate is undertaken using the Rational Method: Q = 2.78 CIA

Where: Q = Peak flow rate (litres/second)
C = Runoff coefficient
I = Rainfall intensity (mm/hour)
A = Catchment area (hectares)

Project Area, A 2.12 hectares
Runoff Coef, C* 0.79

$$I = \frac{A}{(t+B)^c}$$

Where: A, B and C = Parameters defined in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual
I = Rainfall intensity (mm/hour)
t = Time of concentration (minutes)

Return Period (Years)	2	5	10	25	50	100
A	779.0	959.0	1089.0	1234.0	1323.0	1435.0
B	6.0	5.7	5.7	5.5	5.3	5.2
C	0.82	0.80	0.80	0.79	0.78	0.78
T (mins) *	10	10	10	10	10	10
I (mm/hr)	80.1	105.3	121.8	143.0	158.2	174.1
Q (litres/sec)	373.8	491.4	568.7	667.6	738.5	812.8
Q (m3/sec)	0.374	0.491	0.569	0.668	0.738	0.813

* Note recommended value for time of concentration is 10 minutes
as stated in Section 1.1.24.2 of The Town of Milton Engineering and Parks Standards Manual

Long-Term Dewatering Rates: **1.4** **L/s** **(from MCR report dated Jan 2024)**

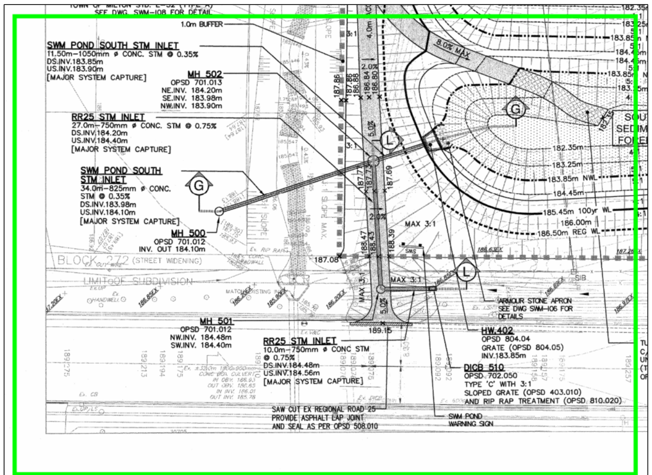
Total Flow Discharging to existing SWM Facility I

Q (litres/sec)	375.1	492.7	570.0	669.0	739.8	814.1

Design Flow that the Existing SWM Facility I was designed to receive (from Page 11 of SWM Calculations)

Q (litres/sec)	374.5	492.4	569.8	669.0	740.0	814.4
Differences (L/s)	-0.6	-0.4	-0.2	0.0	0.2	0.3

Therefore, the Post-Development release rates from the North Block and the holdout property is less than the flow rates that the existing SWM Pond "I" was designed to receive for storm greater than the 25-year storm event. During the smaller storm events, there is a maximum exceedances of 0.6 L/s. The exceedances are considered negligible and therefore, no additional quantity control is required.



Additionally, … The storm sewer was sized for the Mattamy Framgard Subdivsion for an area of 2.13 ha and a runoff coefficient of 0.85. However, the storm sewer is only sized for the 5-year storm event assuming a design flow of approximately 529 L/s. Fortunately, the storm sewers were sized with additional capacity in mind. Similarily, the 750mm storm sewer collecting runoff from Regional Road 25 assuming a drainage area of 1.76 ha and a RC of 0.90. The storm sewer.

The following calculations below will show that the existing storm sewers to SWM Pond "I" will be able to convey the 100-year flow from the North Block and the Regional Road 25 to SWM Pond "I"

Pipe Dia:	825	mm	Drainage Area (A):	2.12	ha	(from WSP ATO)
Slope:	0.35	%	Runoff Coefficient (RC):	0.79	-	(from WSP ATO)
Length:	34	m	A*RC:	1.6748	ha	
n:	0.013	-				
Area:	0.53	m2	During the 5-yr Storm Event:			
Wetted Peri:	2.59	m	Intensity:	105.3	mm/hr	(assuming Tc = 10 min)
			Flow:	489.654	L/s	
Full Velocity:	1.59	m/s	During the 100-yr Storm Event:			
Pipe Capacity:	**0.85**	m3/s	Intensity:	174.1	mm/hr	(assuming Tc = 10 min)
Pipe Capacity:	**849.22**	L/s	Flow:	809.963	L/s	

Pipe Dia:	750	mm	Drainage Area (A):	1.76	ha	(from TMIG Report)
Slope:	0.75	%	Runoff Coefficient (RC):	0.90	-	(from TMIG Report)
Length:	37	m	A*RC:	1.584	ha	
n:	0.013	-				
Area:	0.44	m2	During the 5-yr Storm Event:			
Wetted Peri:	2.36	m	Intensity:	105.3	mm/hr	(assuming Tc = 10 min)
			Flow:	463.107	L/s	
Full Velocity:	2.18	m/s	During the 100-yr Storm Event:			
Pipe Capacity:	**0.96**	m3/s	Intensity:	174.1	mm/hr	(assuming Tc = 10 min)
Pipe Capacity:	**964.13**	L/s	Flow:	766.051	L/s	

Pipe Dia:	1050	mm				
Slope:	0.35	%				
Length:	11.5	m				
n:	0.013	-				
Area:	0.87	m2				
Wetted Peri:	3.30	m	During the 100-yr Storm Event:			
			Flow:	952.761	L/s	(Addition of the Flow)
Full Velocity:	1.87	m/s	During the 100-yr Storm Event:			
Pipe Capacity:	**1.62**	m3/s				
Pipe Capacity:	**1615.52**	L/s	Flow:	1576.01	L/s	(Addition of the Flow)

Water Balance

The water balance can be calculated with a site wide water balance analysis. The water balance is an accounting of the water resources within a given area and can be calculated with the following: P = I + ET + R. Where; precipitation (P) over a given area is equal to the summation of evapotranspiration (ET), surface water runoff (R), and infiltration (I). The following tables summarizes the details for water balance for existing and post-development conditions.

	Precipitation (mm/yr)	Evapotranspiration (mm/yr)	Infiltration (mm/yr)	Runoff (mm/yr)
HydroG report (Per. Area)	807	603	71	133
	100%	75%	9%	16%
HydroG report (Imp Area)	807	161	0	646
	100%	20%	0%	80%

Existing Site Plan

Pervious Area	(%)	(m^3/yr)
Infiltration	9%	1,704
Evapotranspiration	75%	14,474
Runoff	16%	3,192
Precipitation	**100%**	**19,371**

Comments:
Total pervious area = 24003 m²
Based on the pre-development analysis from the Water Balance Report prepared by McClymont & Rak Engineers Inc. dated July 2023

Impervious Area	(%)	(m^3/yr)
Infiltration	0%	0
Evapotranspiration	20%	0
Runoff	80%	0
Precipitation	**100%**	**0**

Total impervious area = 0 m²
Based on the pre-development analysis from the Water Balance Report prepared by McClymont & Rak Engineers Inc. dated July 2023

Proposed Site Plan without LIDs

Pervious Area	(%)	(m^3/yr)
Infiltration	9%	237
Evapotranspiration	75%	2,013
Runoff	16%	444
Precipitation	**100%**	**2,694**

Comments:
Total pervious area = 3338 m²
Based on the pre-development analysis from the Water Balance Report prepared by McClymont & Rak Engineers Inc. dated July 2023

Impervious Area	(%)	(m^3/yr)	
Reuse	0%	0	None proposed at this time
Infiltration	0%	0	No infiltration from impervious surfaces & landscaping on top of underground structure
Evapotranspiration	10%	1,668	1.0 mm of rainfall (equivalent to 10% annual volume) is lost to initial abstraction
Runoff	90%	15,009	Remainder of rainfall leaves site as runoff
Precipitation	**100%**	**16,677**	

Total impervious area = 20665 m² (Assumed 95% Imperviousness)

	Existing				Proposed (no LIDs)			
	Pervious	Impervious	Site-Wide		Pervious	Impervious	Site-Wide	
% Land-Use Coverage	100.0%	0.0%	100.0%		13.9%	86.1%	100.0%	
Reuse (m^3/yr)	0	0	0	0.0%	0	0	0	**0.0%**
Infiltration (m^3/yr)	1,704	0	1,704	8.8%	237	0	237	**1.2%**
Evapotranspiration (m^3/yr)	14,474	0	14,474	74.7%	2,013	1,668	3,680	**19.0%**
Runoff (m^3/yr)	3,192	0	3,192	16.5%	444	15,009	15,453	**79.8%**
Precipitation (m^3/yr)	19,371	0	19,371	100%	2,694	16,677	19,371	100%

As shown, the site would need to infiltrate 1,467 m3 of volume annually to match existing conditions.

Subject

Stormwater Management Criteria - Water Balance - South Block - Catchments 201-204 (Cont.)

Minimum additional Volume Infiltrated / yr	1,467	m³	Difference between existing and proposed conditions
Total Precipitation Volume / yr (impervious)	16,287	m³	Annual Precipitation * Impervious Area contributing to LID
% total average annual rainfall capture	9%		additional rainfall capture by impervious area contributing to LID
Equivalent daily rainfall depth	1	mm	WWFMG Fig 1a
Equivalent Infiltration trench volume	**20.18**	m³	equivalent volume provided by LID
Porosity of Infiltration Trenches	0.40	-	Infiltration trenches to consist of clearstones (porosity of 0.40)

Inflow areas to trenches would need to capture and infiltrate approximately, an additional 1 mm from each rainfall event.

Infiltration trenches located along the NHS Promenade to provide the required infiltration volume.

Trench Number	Catchment	Impervious Area (sq.m)	Runoff Volume (cu. m) (1 mm rainfall event)	Trench Volume Required (m³)	Contingency (15%)	Trench Volume Required With Contingency (m³)	Trench Length (m)	Trench Width (m)	Trench Height Required at Specified Width (m)	Infiltraition Rate At trench Location (mm/hr)	Maximum Allowable Trench Height (m) To Fully Drain in 48 Hours
1	201, 202	10,609	10.61	26.52	3.98	30.50	25	5.00	0.24	5	0.60
2	203, 204	9,573	9.57	23.93	3.59	27.52	25	5.00	0.22	5	0.60
Total		**20,182**	**20.2**	**50**	**8**	**58**					

As shown, the LIDs (sized to capture a 1 mm rainfall event) provide a total storage volume of 20.18 m3, which is equal to the required volume of 20.18 m3.

Total Infiltration trench volume (w/o contingency)	20	m³	Total volume provided by LIDs (without contingency)
Equivalent daily rainfall depth (across whole site)	1.0	mm	LID Vol / total impervious area
% total average annual rainfall capture	**9.0%**		Approx Equivalent rainfall depth from WWFMG Fig 1a

Subject: **Stormwater Management Criteria - Water Balance - South Block - Catchments 201-204 (Cont.)**

Proposed Site Plan with LIDs

Pervious Area	(%)	(m^3/yr)	Comments:
			Total pervious area = 3338 m²
Infiltration	9%	237	Pervious surface estimated to have similar properties to existing conditions in terms of infiltration and ET.
Evapotranspiration	75%	2,013	
Runoff	16%	444	Remainder of rainfall leaves site as runoff
Precipitation	**100%**	**2,694**	

Impervious Area	(%)	(m^3/yr)	Comments:
			Total impervious area = 20665 m²
Reuse	0%	0	None proposed at this time
Infiltration	9%	1,501	Infiltration provided by infiltration trenches
Evapotranspiration	10%	1,668	1.0 mm of rainfall (equivalent to 10% annual volume) is lost to initial abstraction
Runoff	81%	13,508	Remainder of rainfall leaves site as runoff
Precipitation	**100%**	**16,677**	

	Existing				Proposed (with LIDs)			
	Pervious	Impervious	Site-Wide		Pervious	Impervious	Site-Wide	
% Land-Use Coverage	100.0%	0.0%	100.0%		13.9%	86.1%	100.0%	
Reuse (m^3/yr)	0	0	0	0.0%	0	0	0	0.0%
Infiltration (m^3/yr)	1,704	0	1,704	**8.8%**	237	1,501	1,738	**9.0%**
Evapotranspiration (m^3/yr)	14,474	0	14,474	74.7%	2,013	1,668	3,680	19.0%
Runoff (m^3/yr)	3,192	0	3,192	16.5%	444	13,508	13,952	72.0%
Precipitation (m^3/yr)	19,371	0	19,371	100%	2,694	16,677	19,371	100%

As shown, the site with LIDs has increased infiltration when compared to existing conditions.

Water Balance

The water balance can be calculated with a site wide water balance analysis. The water balance is an accounting of the water resources within a given area and can be calculated with the following: P = I + ET + R. Where; precipitation (P) over a given area is equal to the summation of evapotranspiration (ET), surface water runoff (R), and infiltration (I). The following tables summarizes the details for water balance for existing and post-development conditions.

	Precipitation (mm/yr)	Evapotranspiration (mm/yr)	Infiltration (mm/yr)	Runoff (mm/yr)
HydroG report (Per. Area)	807	603	71	133
	100%	75%	9%	16%
HydroG report (Imp Area)	807	161	0	646
	100%	20%	0%	80%

Existing Site Plan

Pervious Area	(%)	(m³/yr)
Infiltration	9%	1,459
Evapotranspiration	75%	12,393
Runoff	16%	2,733
Precipitation	**100%**	**16,585**

Comments:
Total pervious area = 20551 m²
Based on the pre-development analysis from the Water Balance Report prepared by McClymont & Rak Engineers Inc. dated July 2023

Impervious Area	(%)	(m³/yr)
Infiltration	0%	0
Evapotranspiration	20%	97
Runoff	80%	390
Precipitation	**100%**	**488**

Total impervious area = 604 m²
Based on the pre-development analysis from the Water Balance Report prepared by McClymont & Rak Engineers Inc. dated July 2023

Proposed Site Plan without LIDs

Pervious Area	(%)	(m³/yr)
Infiltration	9%	197
Evapotranspiration	75%	1,677
Runoff	16%	370
Precipitation	**100%**	**2,244**

Comments:
Total pervious area = 2780 m²
Based on the pre-development analysis from the Water Balance Report prepared by McClymont & Rak Engineers Inc. dated July 2023

Impervious Area	(%)	(m³/yr)	
Reuse	0%	0	None proposed at this time
Infiltration	0%	0	No infiltration from impervious surfaces & landscaping on top of underground structure
Evapotranspiration	10%	1,483	1.0 mm of rainfall (equivalent to 10% annual volume) is lost to initial abstraction
Runoff	90%	13,346	Remainder of rainfall leaves site as runoff
Precipitation	**100%**	**14,829**	

Total impervious area = 18375 m² (Assumed 95% Imperviousness)

	Existing				Proposed (no LIDs)			
	Pervious	Impervious	Site-Wide		Pervious	Impervious	Site-Wide	
% Land-Use Coverage	97.1%	2.9%	100.0%		13.1%	86.9%	100.0%	
Reuse (m³/yr)	0	0	0	0.0%	0	0	0	**0.0%**
Infiltration (m³/yr)	1,459	0	1,459	8.5%	197	0	197	**1.2%**
Evapotranspiration (m³/yr)	12,393	97	12,490	73.2%	1,677	1,483	3,159	**18.5%**
Runoff (m³/yr)	2,733	390	3,124	18.3%	370	13,346	13,716	**80.3%**
Precipitation (m³/yr)	16,585	488	17,073	100%	2,244	14,829	17,073	100%

As shown, the site would need to infiltrate 1,262 m3 of volume annually to match existing conditions.

Subject

Stormwater Management Criteria - Water Balance - North Block - Catchments 205-208 (Cont.)

Minimum additional Volume Infiltrated / yr	1,262	m³	Difference between existing and proposed conditions
Total Precipitation Volume / yr (impervious)	14,829	m³	Annual Precipitation * Impervious Area contributing to LID
% total average annual rainfall capture	9%		additional rainfall capture by impervious area contributing to LID
Equivalent daily rainfall depth	1	mm	WWFMG Fig 1a
Equivalent Infiltration trench volume	**18.38**	m³	equivalent volume provided by LID
Porosity of Infiltration Trenches	0.40	-	Infiltration trenches to consist of clearstones (porosity of 0.40)

Inflow areas to trenches would need to capture and infiltrate approximately, an additional 1 mm from each rainfall event.

Infiltration trenches located along the NHS Promenade to provide the required infiltration volume.

Trench Number	Catchment	Impervious Area (sq.m)	Runoff Volume (cu. m) (1 mm rainfall event)	Trench Volume Required (m³)	Contingency (15%)	Trench Volume Required With Contingency (m³)	Trench Length (m)	Trench Width (m)	Trench Height Required at Specified Width (m)	Infiltraition Rate At trench Location (mm/hr)	Maximum Allowable Trench Height (m) To Fully Drain in 48 Hours
3	205-208	18,375	18.38	45.94	6.89	52.83	60	5.00	0.18	5	0.60
Total		**18,375**	**18.4**	**46**	**7**	**53**					

As shown, the LIDs (sized to capture a 1 mm rainfall event) provide a total storage volume of 18.38 m3, which is equal to the required volume of 18.38 m3.

Total Infiltration trench volume (w/o contingency)	18	m³	Total volume provided by LIDs (without contingency)
Equivalent daily rainfall depth (across whole site)	1.0	mm	LID Vol / total impervious area
% total average annual rainfall capture	**10.0%**		Approx Equivalent rainfall depth from WWFMG Fig 1a

Proposed Site Plan with LIDs

Pervious Area	(%)	(m³/yr)	Comments:
			Total pervious area = 2780 m²
Infiltration	9%	197	Pervious surface estimated to have similar properties to existing conditions in terms of infiltration and ET.
Evapotranspiration	75%	1,677	
Runoff	16%	370	Remainder of rainfall leaves site as runoff
Precipitation	**100%**	**2,244**	

Impervious Area	(%)	(m³/yr)	Comments:
			Total impervious area = 18375 m²
Reuse	0%	0	None proposed at this time
Infiltration	10%	1,483	Infiltration provided by infiltration trenches
Evapotranspiration	10%	1,483	1.0 mm of rainfall (equivalent to 10% annual volume) is lost to initial abstraction
Runoff	80%	11,863	Remainder of rainfall leaves site as runoff
Precipitation	**100%**	**14,829**	

	Existing				Proposed (with LIDs)			
	Pervious	Impervious	Site-Wide		Pervious	Impervious	Site-Wide	
% Land-Use Coverage	97.1%	2.9%	100.0%		13.1%	86.9%	100.0%	
Reuse (m³/yr)	0	0	0	0.0%	0	0	0	0.0%
Infiltration (m³/yr)	1,459	0	1,459	**8.5%**	197	1,483	1,680	**9.8%**
Evapotranspiration (m³/yr)	12,393	97	12,490	73.2%	1,677	1,483	3,159	18.5%
Runoff (m³/yr)	2,733	390	3,124	18.3%	370	11,863	12,233	71.7%
Precipitation (m³/yr)	16,585	488	17,073	100%	2,244	14,829	17,073	100%

As shown, the site with LIDs has increased infiltration when compared to existing conditions.

APPENDIX

C

Excerpts from the Geotechnical, Geohydrology and Water Balance Reports

C-1 Excerpts from Geotechnical Report (Jul 2023)

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McCLYMONT&RAK
ENGINEERS, INC.
GEO-ENVIRONMENTAL CONSULTANTS

111 ZENWAY BLVD., UNIT 4, VAUGHAN, ONTARIO, L4H 3H9
TEL: 416.675.0160 FAX: 905 851.1722
office@mccrak.com

1271 DENISON ST., UNIT 45, MARKHAM, ONTARIO, L3R 4B5
TEL: 905.470.0160 FAX: 905.475.6371
denison@mccrak.com

McMASTER INNOVATION PARK - COLABORATORY No. 110A
175 LONGWOOD RD. S., HAMILTON, ONTARIO, L8P 0A1
office@mccrak.com

WWW.MCCRAK.COM

G5820 **JULY 2023**

GEOTECHNICAL REPORT
RESIDENTIAL DEVELOPMENT
REGIONAL ROAD 25 AND BRITANNIA ROAD
MILTON, ONTARIO

DISTRIBUTION:

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PREPARED FOR:

MATTAMY HOMES CANADA
7880 KEELE STREET
VAUGHAN, ONTARIO
L4K 4G7

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APPENDICES

1.0 INTRODUCTION

MCR was retained by Mattamy Homes Canada (the Client) to carry out a geotechnical investigation for the proposed residential development located at Regional Road 25 and Britannia Road Milton, Ontario (hereafter referred to as 'the Site').

The objective of the report was to determine design data required for foundations, dewatering, shoring/excavation, backfill, slab on grade and pavement. The above design and construction issues are addressed in the following report.

2.0 SITE CONDITION

The Site is located at the northwestern corner of Regional Road 25 and Britannia Road, in a mixed-use rural, residential and commercial area of the city of Milton, Ontario. The site is irregular in shape with an approximate area of 41,511 m^2.

Etheridge Avenue bisects the Site, running west to east; the southern portion is a vacant lot and the northern portion is occupied by Mattamy Homes office, a parking area and the rest is vacant.

The Site is bounded by a pond to the north, Regional Road 25 to the east, Britannia Road to the south, and a pond/channel to the west.

3.0 PROPOSED DEVELOPMENT

The latest architectural drawings (Appendix A) show the Site is proposed for residential development and will consist of:

- **South Block:** A fifteen [15] storey building (Tower 1) with eight [8] storey podiums, a fourteen [14] storey building (Tower 2) with eight [8] storey podium, a thirteen [13] storey building (Tower 3) with eight [8] storey podiums, and a fifteen [15] storey building (Tower 4) with six [6] storey podium over two [2] levels of underground parking.

- **North Block:** A thirteen [13] storey building (Tower 5) and a twelve [12] storey building (Tower 6), with eight [8] storey podiums over two [2] levels of combined underground parking, and a fifteen [15] storey building (Tower 7) with eight [8] storey podiums over two [2] levels of underground parking.

The finished floor elevations (FFE) at ground level and P2 underground are presented in Table 1 below:

Table 1 – Assumed Finished Floor Depths/Elevations

Building	GF FFE (m)	P2 FFE (m)
Tower 1	186.95	179.00
Tower 2	185.80	178.35
Tower 3	185.60	177.70
Tower 4	184.50	177.05
Tower 5	188.15	180.70
Tower 6	188.15	180.70
Tower 7	188.25	180.80

4.0 SITE INVESTIGATION

Initially, twelve boreholes (BH 1 to BH 12), were drilled by Shad & Associates Inc., in February and March 2018 to depths of 7.80 to 8.10 m.

In addition, nine boreholes (BH 101 to BH 109), were drilled by MCR in December 2022 and January 2023 to depths of 7.30 to 21.40 m.

Due to the presence of boreholes by Shad & Associates Inc., sampling in boreholes 102, 103, 106 and 108 started at a depth of 9.15 m and continued to maximum explored depth of the boreholes.

All boreholes by Shad & Associates Inc., except boreholes 2, 6, 7 and 11, were equipped with monitoring wells for long-term groundwater monitoring and sampling.

Location of the boreholes are shown on Drawing No. 1 and Borehole logs by MCR and Shad & Associates Inc., are presented in Appendices B and C, respectively.

Soil samples were taken using the Standard Penetration Test (SPT) method and were placed in clean, sealed plastic bags in the field and transported back to our laboratory where they were further examined for soil characterization.

Moisture contents of most of soil samples and grain size analyses (soil gradation), for selected soil samples, from different boreholes, were determined and the results are presented in Appendix B.

In addition, selected samples were transported to Bureau Veritas to be tested for common corrosion parameters, including pH, resistivity, oxygen reduction potential (redox), chlorides and sulphate content. The laboratory test results are presented in Appendix D.

MCR borehole elevations, referred to in this report, are geodetic and metric and are interpolated from survey plans by R-PE Surveying Ltd. dated February and March 2018.

5.0 SOIL AND GROUNDWATER CONDITIONS

Subsurface conditions encountered at the borehole locations are shown on Borehole Log Sheets, attached in Appendices B&C, and summarized on a Soil Profile/Drawing No. 2 to 5, as follows:

Fill: Compact fill material was encountered at the surface of all boreholes. The fill material extended to depths ranging from 0.4 to 0.9 m. The fill consisted of silty sand/sandy silt/clayey silt/silty clay, sand and gravel soils. The brown/dark brown to reddish brown fill was in a moist condition and contained some to trace of organics, clay, gravel, and rootlets.

For the purpose of offsite disposal, the type/quantity and extent of the existing fill layer should be explored by further test pit investigation, prior to contract award.

Silty Sand/Sandy Silt: A dense silty sand/sandy silt till layer was encountered below the fill in boreholes 104, 105, 107 and 109. The brown silty sand/sandy silt layer was in a moist condition and contained traces of clay. The silty sand/sandy silt layer extended to the full depth of borehole 104 and a depth of 2.30 m in boreholes 105, 107 and 109.

Clayey Silt/Silty Clay (Till): A very stiff to hard clayey silt/silty clay till layer was encountered below the fill and silty sand/sandy silt layer in all boreholes (except 102, 103, 106 and 108). The reddish brown to grey clayey silt/silty clay till layer was in a moist to wet condition and contained some to trace of sand, gravel and shale fragments. The clayey silt/silty clay till layer extended to the full depth of boreholes 2, 3, 5, 8, 11 and 109 and to depths ranging from 4.55 to 10.65 m in all other boreholes.

Sand and Gravel/Silty Sand/Sandy Silt (Till): A very dense sand and gravel/silty sand/sandy silt till deposit was observed below the clayey silt/silty clay till layer in all boreholes. The brown to reddish brown sand and gravel/silty sand/sandy silt (till) deposit was in a moist to wet condition and contained traces of clay, gravel and shale fragments. The sand and gravel/silty sand/sandy silt till layer extended to a depth of 18.30 m in borehole 101 and to the full depth of all other boreholes.

Clayey Silt Till: A hard layer of clayey silt till was detected below the sand and gravel/silty sand/sandy silt till deposit in borehole 101. The reddish brown layer was in a moist condition and contained traces of sand, gravel and shale fragments. The clayey silt till layer extended to the full depth of borehole exploration.

It should be noted that the silt/clay/sand/till soil is unsorted deposit; therefore, boulders and cobbles are anticipated.

Groundwater: Upon completion of drilling all monitoring wells by Shad and Associates Inc., were dry.

The results of water level readings are summarized on the Record of Borehole Sheets in Appendices B&C and Table 2.

Table 2 – Groundwater Level Monitoring Results

Monitoring Well Id	Ground Surface Elevation (masl)	Water Level (mbgs)	Groundwater Elevation (masl)	Date of Measurement (mm/dd/yyyy)	Depth of Well (mbgs)	Depth of Bentonite (mbgs)	Length of Screen (m)	Inside Diameter of Pipe (mm)	Top of Monitoring Well
BH 1	184.70	2.80	181.90	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		2.90	181.80	3/16/2018					
		2.80	181.90	1/6/2023					
BH 3	185.80	3.70	182.10	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		3.60	182.20	3/16/2018					
		3.74	182.06	1/6/2023					
BH 4	185.10	3.60	181.50	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		3.50	181.60	3/16/2018					
		3.26	181.84	1/6/2023					
BH 5	186.60	4.20	182.40	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		4.30	182.30	3/16/2018					
		0.74	185.86	1/6/2023					
BH 8	186.70	DRY	-	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		6.40	180.30	3/16/2018					
		NF	-	1/6/2023					
BH 9	186.70	2.90	183.80	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		2.90	183.80	3/16/2018					
		3.76	182.94	1/6/2023					
BH 10	186.60	2.90	183.70	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		3.00	183.60	3/16/2018					
		2.94	183.66	1/6/2023					
BH 12	186.80	3.60	183.20	3/9/2018	7.70	5.70	3.05	50	Flush Mount
		3.60	183.20	3/16/2018					
		3.72	183.08	1/6/2023					
Min	184.70	0.74	180.30	-	7.70	-	-	-	-
Max	186.80	6.40	185.86	-	7.70	-	-	-	-
Average	186.13	3.40	182.67	-	7.70	-	-	-	-

It should be noted that groundwater levels are subject to seasonal fluctuations. Consequently, definitive information on the long-term groundwater levels could not be obtained during this investigation.

Subject to the owner's approval, groundwater monitoring should continue, and the results should be presented in a separate report addressing Geohydrology/Dewatering induced Settlement issues.

A Geohydrology assessment dated January 2023 was completed by MCR and results are presented in a separate report.

6.0 FOUNDATION

The latest architectural drawings (Appendix A) show that the Site is proposed for residential development and will consist of:

- **South Block:** A fifteen [15] storey building (Tower 1) with eight [8] storey podiums, a fourteen [14] storey building (Tower 2) with eight [8] storey podium, a thirteen [13] storey building (Tower 3) with eight [8] storey podiums, and a fifteen [15] storey building (Tower 4) with six [6] storey podium over two [2] levels of underground parking.

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The P2 finished floor elevations (FFE) in Towers 1 to 7, range between 180.80 to 177.05 m.

The following recommendations are based on the current information and design. Should changes be made during the design phase or construction, this office must be informed and retained to modify recommendations accordingly or propose additional field work.

Subject to design loads/grades the proposed residential development with two [2] levels of U/G parking, can be supported by conventional spread/strip footings, founded in the competent undisturbed (by hydrostatic pressure) native soils.

6.1 SPREAD/STRIP FOOTINGS

The proposed footings could be proportioned using the following bearing resistance:

Factored Bearing Resistance at ULS = 560 kPa
Bearing Resistance at SLS = 400 kPa

When the underside of the proposed footings is founded at or below at or below Elevation of 179.90 m, subject to field inspection and confirmation during excavations.

6.2 GENERAL FOUNDATION NOTES

It is essential that the groundwater be lowered a minimum of 1.0 m below the underside of the proposed footings/elevator pit. The clayey silt/sandy silt soil encountered at the foundation level, will be subject to dilation/quick condition when saturated/subjected to hydrostatic pressure, subject to groundwater monitoring results.

We request that a preliminary foundation plan be prepared. Our office must review the foundation plan and detailed settlement analyses must be carried out for the highest column load/bearing resistance combination.

The proposed settlement analyses will quantify the anticipated amount of the "during" and "post construction' settlement. The actual amount of settlement should be monitored during the construction of the buildings.

It should also be noted that the till, and interbedded sand soils, in southern Ontario are glacial/interglacial in origin and as such contain cobbles, boulders and other erratic rock, the precise placement and location of which cannot be determined without comprehensive excavation. Removal of cobbles, boulders and other erratic rock will usually result in extra excavation and construction cost.

It is recommended that your excavation and construction contract provisions include unit prices for excavation into soils which may contain cobbles, boulders and erratic rock to minimize potential unexpected extra costs during excavation and foundation installations.

In case of water penetration through the exposed shoring toes (within the waterbearing sand deposit/wet silty soils), bentonite mud, tremie concrete and/or re-drillable low strength concrete may have to be used. The contractor must be prepared to deal with the situation without undue delays.

Adjacent footings, founded at different elevations, should be stepped at 10 horizontal to 7 vertical.

For frost protection requirements, all foundations in unheated underground parking P2 must have a minimum soil cover of 0.90 m.

Any water or loose materials must be removed from the footing bases prior to placing concrete.

The recommended resistance at SLS allows for up to 25 mm of total settlement. Potential differential settlements are to be evaluated after completion of the foundation drawings.

Furthermore, the recommended bearing resistance and foundation elevations have been calculated from the borehole information and, are intended for design purposes only.

More specific information with respect to soil/foundation conditions between the boreholes will be available when the proposed foundation installation is underway. Therefore, the encountered soil/foundation conditions must be verified in the field, and all foundations must be inspected and approved by our office prior to placement of concrete.

As indicated on Drawing No. 6, there is a 9 m wide buffer between the shoring line and the property boundary. Additionally, the existing slope towards the Natural Heritage System (N.H.S.), has a very gentle inclination of 4V:34H. Based on this assessment, it is anticipated that the underground parking structure will have no discernible impact on the N.H.S.

7.0 EARTHQUAKE CONSIDERATION

The building must be designed to resist a minimum earthquake force. The National Building Code specifies that the building be designed to withstand a minimum lateral seismic force, V, which is assumed to act non-currently in any direction on the building as per the following expression:

$$V = S(T_a)\, M_v I_E W / R_d R_o$$

It should be noted that V shall not be less than:

$$S(2.0)\, M_v I_E W / R_d R_o$$

In addition, the SFRS (Seismic Force Resisting System (s)) with R_d equal to or greater than 1.5, V should not be greater than:

$$2/3\, S(0.2)\, I_E W / R_d R_o$$

Where $S(T_a)$ shall be calculated by $S_a(T_a)F_a$ or $S_a(T_a)F_v$, depending on fundamental lateral period T_a. The terms, which are relevant to the geotechnical conditions at the site, are acceleration-based site coefficient F_a and velocity-based site coefficient F_v.

For the subject site, which is classified as Class C (based on the borehole information), the applicable values of F_a and F_v are 1.0 and 1.0, respectively. A structural consultant should review all factors.

To better define/confirm the site classification a Shear Wave Velocity (SWV) test must be carried out.

8.0 BASEMENT WALLS

Underground parking walls should be designed to resist a pressure "p", at any depth, "h" below the surface, as given by the expression:

$$p = K[\gamma h + q]$$

Where: K = 0.40 is the earth pressure coefficient considered applicable

γ = 21.7 kN/m^3 is the unit weight of backfill

q = an allowance for surcharge.

The above equation assumes that perimeter drains will be provided and that the backfill against subsurface walls, where applicable, would be a free draining granular material.

However, subject to groundwater conditions and the presence of the wet sandy silt/ silty sand soils, all subject to further groundwater monitoring results, we suggest that perimeter walls below the groundwater level be designed for hydrostatic pressure to resist a pressure "p", at any depth "h" below the surface, as given by the expression:

$$p = \begin{cases} Kq + K\gamma_m h & h \leq D_w \\ Kq + K\gamma_w D_w + K(\gamma_s - \gamma_w)(h - D_w) + \gamma_w(h - D_w) & h > D_w \end{cases}$$

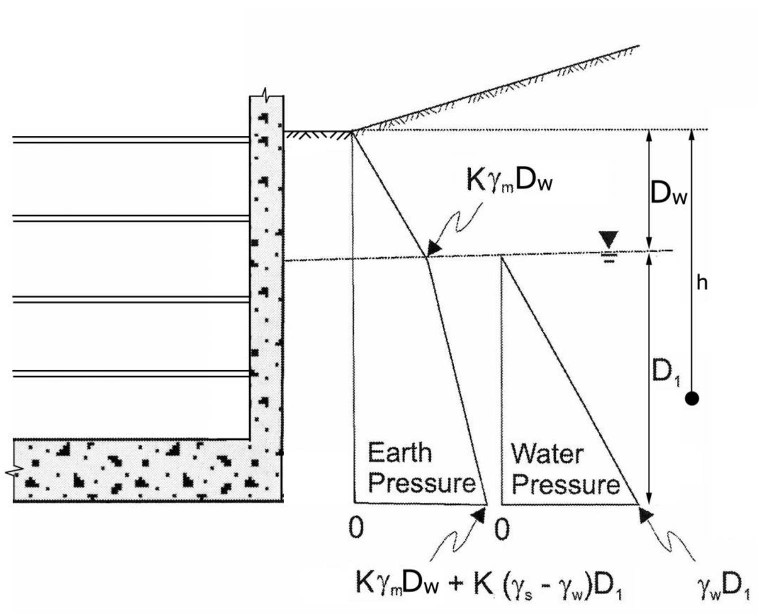
Where: K = 0.50 is the earth pressure coefficient considered applicable

γ_m = 20 kN/m^3 is moist or wet soil unit weight

γ_s = 21.7 kN/m^3 is saturated soil unit weight

γ_w = 9.80 kN/m^3 is the unit weight of water

q = an allowance for surcharge



9.0 DEWATERING

The excavation for the proposed underground parking will extend below the groundwater table.

In order to protect the bottom and sides of the excavation from being disturbed by excess groundwater pressure, i.e. to prevent quick sand/dilating silt conditions, the water table must be lowered to at least 1.0 m below the bottom of the footing/elevator excavations.

Positive dewatering, such as well points/eductors will be required for the proposed excavation, subject to long term groundwater monitoring results and depth of excavation.

The selected dewatering system, designed and installed by a specialty contractor, will be most effective if it is installed and activated at the earliest opportunity during general excavation.

The selected dewatering contract must be performance driven and the contractor must provide a performance bond. In addition, upon completion of system's installation, the contractor must produce a written statement that "The system installed is robust enough to lower and maintain groundwater at least 1.0 m below the lowest footing elevation, without impacting the integrity of shoring or foundation soils.

It is reiterated that on site soils might be subject to localized piping. Creation of piping channels might result in a substantial increase in the volume of both temporary dewatering and permanent drainage. It is critical that upon completion of general excavation **potential formation of localized piping be carefully evaluated and appropriate corrective measures implemented.**

A pre-construction survey of adjacent structures/roads should be carried out prior to the dewatering/shoring construction stage. Potential adverse effects on adjacent structures, due to the dewatering must be assessed/quantified and suitable preventive/remedial measures implemented.

10.0 EXCAVATION AND BACKFILL

Excess soils shall be managed in accordance to O. Reg. 406/19. As of January 1, 2022, the Project Leader may be required to file a notice in the registry as prescribed under Section 8 of the regulation. The notice shall contain the information set out in Schedule 1 of the regulation. Before the notice is filed the Project Leader shall ensure that a Qualified Person (Qualified Person within the meaning of Section 5 or 6 of O. Reg. 153/04) prepares the documents, as required, under Sections 11, 12, 13 of the regulation.

The Project Leader shall, if required to file a notice and before removing excess soil from the project area, develop and apply a tracking system in accordance with the Soil Rules, to track each load of excess soil during its transportation and deposit.

No major problems will be encountered for the anticipated depth of general excavations, carried out within a shoring wall enclosure.

For excavation above the water table, the anticipated water seepage, if any, into the excavations from the more permeable seams/lenses or surface run-off can be handled by conventional pumping methods.

A dewatering system such as wellpoints/eductors will be required for excavation at/below the groundwater level, subject to long term groundwater monitoring results.

The material to be used for backfilling in the service trenches should be suitable for compaction, i.e. free of organics and with natural moisture content, which is within 2% percent of the optimum moisture content. The backfill material should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

The backfill under floor slab and against the subsurface walls, where applicable, should be free draining granular fill, preferably conforming to the Ontario Provincial Standard Specification for granular base course, Granular B.

11.0 SHORING

A shoring system should be designed to protect adjacent structures, roads and services. The fourth edition of the Foundation Manual should be referred to for the design of the shoring system.

It should be noted that groundwater and boulders may be encountered during soldier pile/caisson construction, and the contractor must be prepared to deal with boulders and water seepage into the caisson shafts without undue delays.

Due to the groundwater and wet silty/sandy soil conditions, it will be difficult to prevent groundwater from penetrating into the excavation through gaps in timber lagging.

The geotechnical parameters, which are considered to be applicable for the design, are as follows:

Active earth pressure coefficient Ka = 0.45 for walls in areas where structures or sensitive services are being supported.

Active earth pressure coefficient Ka = 0.28 for remaining areas.

Natural unit weight of soil = 21.7 kN/m^3

Any surcharge loads must be included in the lateral pressure calculations.

Lateral movements of the shoring wall, designed using Ka = 0.28, are expected to be in order of 15 mm. They are expected to be less if Ka value of 0.45 is used. The expected movements are based on a properly constructed system.

The horizontal and vertical movements should be monitored during construction to ensure a satisfactory performance of the shoring system.

The soil anchors should be designed for 35 kPa, subject to confirmation by at least two load tests. **It is re-iterated that subsurface conditions may vary beyond the site's confines**. As a result, the design values must be confirmed by at least two load tests, carried out to twice the design load.

It is imperative that a stability analysis of the entire support system is undertaken prior to commencement of the shoring construction. Our office should review the final shoring design.

The shoring system and surrounding structures must be monitored for horizontal and vertical movements, prior to, during and after the excavation.

Again, a pre-construction survey of the surrounding structures roads is recommended prior to commencement of shoring construction.

In addition, the shoring system and surrounding structures must be monitored for horizontal and vertical movements, prior to, during and after the excavation.

12.0 SLAB ON GRADE AND PERMANENT DRAINAGE

In case of PWDS/infiltration gallery alternative is adopted and approved by the City and the MECP/ECA, the lowest garage floor slab can be constructed as slab on grade (SOG), supported by competent native undisturbed sand/silt soils.

Any soft spots revealed during proof-rolling should be sub-excavated and backfilled with suitable granular material, compacted to 98% SPMDD.

Upon completion of foundation work, the SOG should rest on a well compacted bed of size 19 mm clear stone at least 200 mm thick. The stone bed would act as a barrier and prevent capillary rise of moisture from the subgrade to the floor slab.

Subject to permits, a permanent Private Water Drainage System (PWDS), as shown on Drawing No. 7 and 8, where shoring is constructed, could be considered. Please note that MCR does not prepare working/shop drawings for the PWDS.

To minimize siltation, all drainage pipe connections must be solid slotted PVC, with elbows and Ts, no “butt” end connections should be permitted. The pipes should slope to a sump at a minimum 1% slope.

Perimeter drainage pipes, with a positive gravity outlet, should be solid and slotted

PVC with a minimum of 0.5% slope. In addition, silt traps must be provided at convenient/accessible locations.

We request that PWDS drawings indicate design elevations for both perimeter and underfloor installation. MCR will provide calculations for sizing of permanent pumps, when required.

Upon completion of general excavation, scope and adequacy of the PWDS is to be re-evaluated. The installation of PWDS must be inspected by our office, prior to placement of filter stone.

Any design changes must be approved by the architect and reflected on mandatory as built drawings.*

* A copy of this page “Slab on grade and Permanent Water Drainage System” page should be posted at a site office as a permanent display.

In addition, the elevator pit should be fully waterproofed as shown on Drawing No. 9.

13.0 PAVEMENT

The critical section of pavement will be at the transition from the infinitely rigid substructure onto soil/backfill subgrade.

As a result, we suggest that an approach type slab be considered to protect underground utilities (on the City’s property) at the entrance/exit points, as shown on Drawing No. 10.

The approach slab will alleviate detrimental effects of dynamic loading/settlement/pavement depression in the backfill to the rigid substructure.

All granular materials used in the pavement construction should be compacted to 100% of the Standard Proctor Maximum Dry Density.

Asphaltic concrete layer should be compacted to the range of 92 to 96.5% of maximum relative density.

Pavement structures presented in tables 4 and 5 are typical. Subject to the anticipated road traffic volumes/AADT/axle loads, the pavement structural design matrix as per Town of Milton Standards, must be followed.

Table 4 – Typical Pavement Structure

Pavement Layer	Recommended Thickness for Light Duty Parking	Recommended Thickness for Heavy Duty Parking
Asphaltic Concrete	40 mm OPSS HL 3 40 mm OPSS HL 8	50 mm OPSS HL 3 75 mm OPSS HL 8
OPSS Granular A Base (or 19mm Crushed Limestone)	150 mm	150 mm
OPSS Granular B (or 50mm Crushed Limestone)	200 mm	350 mm

Table 5 – Typical Composite Pavement Structure

Pavement Layer	Compaction Requirements	Heavy Duty Pavement
Asphaltic Concrete	92 to 96.5% of Maximum Relative Density	50 mm OPSS HL 1 or HL 3
Portland Cement Concrete (CAN3-CSA A23.1) - Class C-2	CAN3-CSA A23.1	150 mm
Base Course: Granular A (OPSS 1010) or 19 mm Crusher Run Limestone	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm

A typical pavement structure above garage roof slab, please see Drawings No. 11 & 12.

14.0 CHEMICAL PROPERTIES OF THE SOIL

Two (2) samples from boreholes 102 and 106 were submitted to Bureau Veritas to be tested for common corrosion parameters, including pH, resistivity, oxygen reduction potential (redox), chlorides, sulfides and sulphate content. The laboratory test results are presented in Appendix D.

14.1 CORROSIVITY

The results regarding corrosivity of the subsurface soil and the corresponding points based on American Water Works Association (AWWA) document, "Polyethylene Encasement for Ductile-Iron Pipe Systems" ANSI/AWWA C105/A21.5-18, dated December 1, 2018, are presented in Table 6.

Table 6 – Results of Soil Corrosivity Potential

Sample ID	Depth (m)	Parameter	Measured Value	ANSI/AWWA Point Rating	Total ANSI/AWWA Points
BH102 SS10	10.70	Sulphide (%)	<0.00005	2	3
		pH	8.04	0	
		Resistivity (ohm.cm)	5000	0	
		Redox Potential (mV)	350	0	
		Moisture (%)	10	1	
BH106 SS9	9.15	Sulphide (%)	<0.00005	2	3
		pH	8.03	0	
		Resistivity (ohm.cm)	3700	0	
		Redox Potential (mV)	250	0	
		Moisture (%)	11	1	

According to AWWA a value below 10 for total points is considered non-corrosive to ductile-iron pipes and therefore no corrosion protection is recommended. It should be noted that the analytical results only provide an indication of the potential for corrosion.

14.2 SULPHATE ATTACK

The concentration of water-soluble sulphate content of the tested samples was 0.0073% and 0.0110% which are below the CSA Standard of 0.1% water-soluble sulphate (Table 3 - Additional Requirements for Concrete Subjected to Sulphate Attack from Canadian Standard CSA A23.1). Therefore, no particular protection measure, such as special concrete mix, against sulphate attack needs to be implemented.

15.0 GENERAL COMMENTS

The comments given in this report are intended only as guidance for design engineers and are subject to field verification during construction. As more specific subsurface information, with respect to conditions between boreholes becomes available during excavations on the subject site, this report should be updated.

Contractors bidding on or undertaking the work should decide on their own investigations, as well as their own interpretations of the factual borehole results. This concern specifically applies to the classification of the subsurface soil and the potential reuse of these soils on/off site.

The contractors must draw their own conclusions as to how the near surface and subsurface conditions may affect them.

We trust this report contains information requested at this time. However, if any clarification is required or if we can be of further assistance, please call us.

Respectfully,
McClymont & Rak Engineers Inc.

LICENSED PROFESSIONAL ENGINEER
L. J. RAK
38011011
PROVINCE OF ONTARIO

S. Tavassoli, M.Sc., E.I.T.

L.J. Rak, M.Eng., P.Eng.

C-2 Excerpts from Geohydrology Report (Jan 2024)

MCR ENGINEERS LTD.
GEO-ENVIRONMENTAL CONSULTANTS

111 ZENWAY BLVD., UNIT 4, VAUGHAN, ONTARIO, L4H 3H9
TEL: 416.675.0160 FAX: 905.851.1722
office@mccrak.com

1271 DENISON ST., UNIT 45, MARKHAM, ONTARIO, L3R 4B5
TEL: 905.470.0160 FAX: 905.475.6371
denison@mccrak.com

WWW.MCCRAK.COM

G5820 **JANUARY 2024**

PRELIMINARY GEOHYDROLOGY ASSESSMENT NORTHWESTERN CORNER OF REGIONAL ROAD 25 AND BRITANNIA ROAD MILTON, ONTARIO

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PREPARED FOR:

MATTAMY (MILTON WEST) LIMITED
3300 BLOOR STREET WEST, SUITE 1800
TORONTO, ONTARIO
M8X 2X2

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1.0 INTRODUCTION

Mattamy (Milton West) Limited (the Client) intends to redevelop the property located at North-western corner of the intersection of Regional Road 25 and Britannia Road, Milton, Ontario (hereafter referred to as 'the Site'). MCR Engineers Ltd. (MCR) was retained to conduct a Geohydrology Assessment for the Site to evaluate the temporary dewatering and permanent drainage in relation to the proposed redevelopment.

1.1 SCOPE OF WORK

The objectives of the Geohydrology Assessment are to determine the following:

- Hydrogeological conditions of the Site, including the groundwater and phreatic surface, subsurface elevations and flow patterns and the interaction with the design and construction of the proposed development.
- Reviewing the available background information for the Site obtained from MCR's files, and architectural drawings.
- Estimate the potential temporary dewatering flow rates during construction and assessment of potential impacts on the surrounding environment.
- Estimate the long term flow rates from the Private Water Drainage System (PWDS) of the proposed building.
- Assess the permitting requirements for both dewatering and discharge with the Ministry of Environment, Conservation and Parks (MECP) and the Municipality of Halton (the City), respectively.
- Summarize the findings in a Geohydrology Assessment Report.

1.2 SITE DESCRIPTION

The Site is located at the northwestern corner of Regional Road 25 and Britannia Road, in a mixed-use rural, residential and commercial area of the city of Milton, Ontario. The site is irregular in shape with an approximate area of 41,511 m^2.

The Site is bounded by a pond to the north, Regional Road 25 to the east, Britannia Road to the south, and a pond/channel to the west. Etheridge Avenue bisects the Site, running west to east. The Site is presently a vacant lot.

Currently the Site does not have a Legal description. The topographic surveys are attached in Appendix A.

1.3 PROPOSED DEVELOPMENT

The Site is proposed for residential development (Appendix B) and will consist of:

- **North Block:** A fifteen [15] storey building (Building 5), a twelve [12] storey building (Building 6), and a fourteen [14] storey building (Building 7) over two [2] levels of underground parking.
- **South Block:** A fifteen [15] storey building (Building 1), a thirteen [13] storey building (Building 2), an eleven [11] storey building (Building 3), and a fourteen [14] storey building (Building 4) over two [2] levels of underground parking.

The design grades are not known at this stage. Therefore, our recommendations should be considered preliminary. Our suggestions might be revised when the proposed Site/Foundation Plan becomes available.

However, the assumed finished floor elevation (FFE) at ground level is expected to be at an elevation of 187.65 to 188.0 meters above sea level (masl). The P2 FFE will be at an approximate elevation of 179.70 to 180.50 masl for the North Block and at 181.05 masl for the South Block.

Presently it is assumed that the proposed buildings will be supported by conventional spread/strip footings founded in silty to sandy/clayey silt soils. The size of the shoring play layout was assumed to cover approximately:

- **North Block:** 165 m by 80 m
- **South Block:** 210 m by 82 m

A sub-floor Private Water Drainage System (PWDS) with perimeter weeping tile will be required for the proposed development. A soldier pile and lagging shoring system is expected for temporary excavation.

1.4 PROPERTY OWNERSHIP

The Site is owned and intended for redevelopment by Mattamy (Milton West) Limited. The Owner is represented by Ms. Christine Chea, with the following contact information:

Ms. Christine Chea, MCIP, RPP
Direction, Development, GTA Urban
3300 Bloor Street West, Suite 1800
Toronto, Ontario
M8X 2X2
Email: christine.chea@mattamycorp.com

1.5 REVIEW OF PREVIOUS REPORTS

The following geo-environmental reports were provided for review prior to initiating the investigation:

- Shad & Associates Inc. report titled, *Geotechnical Investigation Report, Proposed Residential Condominium Development, Framgard Property – Major Node, Regional Road 25, North of Britannia Road, Milton, Ontario,* prepared for Mattamy Willmott Limited, dated March 2018.
- MCR report titled, *Geotechnical Report, Residential Development, Regional Road 25 and Britannia Road, Milton, Ontario,* prepared for Mattamy (Milton West) Limited, dated January 2024.

2.0 HYDROGEOLOGICAL CONDITIONS

2.1 PHYSICAL SETTING

The Site is located in the Town of Milton and is situated in a mixed-use rural, residential, and commercial area. The nearest major intersection is Regional Road 25 and Britannia Road, located southeast of the Site. A branch of The West Tributary of the Sixteen Mile Creek is located approximately 30 m west of the Site.

The Site is located at an elevation of approximately 184 to 186 m above sea level (asl) and the topography across the Site slopes from the north to south. The surrounding area slopes from northwest to southeast, towards the Sixteen Mile Creek.

The Site is bounded by the following properties/features:

North	A pond
South	Britannia Road
East	Regional Road 25
West	Pond/Channel

2.2 TOPOGRAPHY

According to the topographic map, published by the Government of Canada; Natural Resources Canada at the Government of Canada website: http://atlas.gc.ca/toporama/en/index.html, the ground surface at the Site slopes from north to south and the surrounding area sloping from northwest to southeast towards the Sixteen Mile Creek.

2.3 REGIONAL GEOLOGY AND HYDROGEOLOGY

According to the geological map entitled "Quaternary Geology of Ontario, Southern Sheet", published by the Ontario Ministry of Development and Mines, dated 1991, the overburden in the study area consists mainly of Halton till, predominantly silt and clay, minor sand, basin and quiet water deposits. Groundwater flow is expected to be directed southeast towards the Sixteen Mile

Creek.

According to the Ontario Ministry of Development and Mines, Map No. 2554 “Bedrock Geology of Ontario, Southern Sheet, 1991”, the bedrock typically consists of Upper Ordovician shale, limestone, dolostone and siltstone Queenston Formation. On a regional scale, groundwater is expected to flow south-east, towards the Sixteen Mile Creek.

2.4 LOCAL GEOLOGY AND HYDROGEOLOGY

On a local scale, geological conditions and hydrogeology are similar to the ones at a regional scale. Locally, near surface groundwater flow may be influenced by underground structures (e.g., service trenches, catch basins, and building foundations or surface watercourses). No surface water features are present onsite and there are no Provincially Significant Wetlands in the vicinity of the Site.

3.0 SCOPE OF INVESTIGATION

3.1 OVERVIEW OF SITE INVESTIGATION

- Initially, twelve boreholes (BH 1 to BH 12) were drilled by Shad & Associates Inc. from February to March 2018 to depths ranging from 7.80 to 8.10 m.
- Nine boreholes (BH 101 to BH 109) were drilled by MCR in December 2022 to January 2023 to depths ranging from 7.30 to 21.40 m.
- Boreholes 1, 3 to 5, 8 to 10 and 12 were equipped with monitoring wells for long-term groundwater monitoring and sampling.
- The borehole locations are shown in Drawing No. 1 and the records are presented in Appendix C.
- Groundwater levels were recorded from all available monitoring wells over various dates and the data is presented in Table 1.
- Groundwater samples were collected from BH 1 and 10 in December 2022 for chemical analysis of the Municipality of Halton Sewers By-Law criteria.

3.2 MONITORING WELL INSTALLATION

It is assumed that all monitoring wells by Shad and Associates Inc. were installed with a 50 mm diameter schedule, 40 PVC pipe and a 3.05 m long slotted well screen. Well screens were surrounded by a silica sand pack to at least 0.6 m above the top of screen with a bentonite seal extending from above the sand pack to within 0.5 m of the ground surface. All monitoring wells were completed with a flush mounted cover at ground surface. Monitoring well installation was done in accordance with the *Ontario Water Resources Act*, Sections 35 to 50.

3.3 ELEVATION SURVEYING

MCR elevations referred to in this report are metric and geodetic and are interpolated from the provided topographic survey prepared by Rady-Pentek & Edward Surveying Ltd., dated February 9 and April 13, 2018. Borehole elevations are shown on the borehole logs in Appendix C.

3.4 GROUNDWATER SAMPLING

All groundwater sampling activities were conducted in accordance with Ontario Regulation (O.Reg.)153/04, as amended to O.Reg.511/09, July 2011. All monitoring wells were developed prior to sampling activities using a Waterra Hydrolift II (HL-1217) inertial lift pump by purging at least three well volumes or until the monitoring well was purged dry. Groundwater samples were obtained at least 24 hours' post-development under static conditions. No samples were field filtered prior to laboratory analysis, in accordance with the standard.

3.5 GROUNDWATER ANALYSIS

A groundwater sample collected in December was submitted to ALS Laboratory Group (ALS) of Richmond Hill, Ontario, certified by the Canadian Association for Laboratory Accreditation (CALA), for chemical analysis. The Certificates of Analysis received are included in Appendix D. The contact information for the laboratory used is included below.

ALS Laboratory Group
95 West Beaver Creek Road
Richmond Hill, ON L4B 1H2

All groundwater samples were submitted for bulk chemical analysis for the criteria provided in the *Ontario Halton Sanitary Sewer By-Law No. 02-03 (March 2003)*. The results of chemical analysis were compared to the criteria provided in *Table 1 – Limits for Sanitary and Combined Sewers Discharge and Table 2 – Limits for Storm Sewer Discharge*. These guidelines establish the maximum allowable concentrations of specific analytical parameters for water discharged into either the municipal sanitary and/or storm sewer system respectively.

4.0 INVESTIGATION RESULTS

4.1 GEOLOGY

The ground surface elevation across the Site varies from 187.50 masl (BH 104) to 184.70 masl (BH 1). Based on the investigations by MCR and Shad and Associates Inc., the geologic formations beneath the Site are illustrated in borehole logs (Appendix C) and include the following (from surface to depth):

Please note that boreholes 102, 103, 106 and 108 were straight drilled to 9.15 m due to proximity to Shad and Associates Inc. boreholes.

Fill: Compact fill material was encountered at the surface of all boreholes. The fill material extended to depths ranging from 0.4 to 0.9 m. The fill consisted of silty sand/sandy silt/clayey silt/silty clay, sand and gravel soils. The brown/dark brown to reddish brown fill was in a moist condition and contained some to trace of organics, clay, gravel, and rootlets.

For the purpose of offsite disposal, the type/quantity and extent of the existing fill should be explored by further test pit investigation prior to general excavation (prior to contract award).

Silty Sand/Sandy Silt: A dense silty sand/sandy silt till layer was encountered below the fill in boreholes 104, 105, 107 and 109. The brown silty sand/sandy silt layer was in a moist condition and contained traces of clay. The silty sand/sandy silt layer extended to the full depth of borehole 104 and a depth of 2.30 m in boreholes 105, 107 and 109.

Clayey Silt/Silty Clay (Till): A very stiff to hard clayey silt/silty clay till layer was encountered below the fill and silty sand/sandy silt layer in all boreholes (except 102, 103, 106 and 108). The reddish brown to grey clayey silt/silty clay till layer was in a moist to wet condition and contained some to trace of sand, gravel and shale fragments. The clayey silt/silty clay till layer extended to the full depth of boreholes 2, 3, 5, 8, 11 and 109 and to depths ranging from 4.55 to 10.65 m in all other boreholes.

Sand and Gravel/Silty Sand/Sandy Silt (Till): A very dense sand and gravel/silty sand/sandy silt till deposit was observed below the clayey silt/silty clay till layer in all boreholes. The brown to reddish brown sand and gravel/silty sand/sandy silt (till) deposit was in a moist to wet condition and contained traces of clay, gravel and shale fragments. The sand and gravel/silty sand/sandy silt till layer extended to a depth of 18.30 m in borehole 101 and to the full depth of all other boreholes.

Clayey Silt Till: A hard layer of clayey silt till was detected below the sand and gravel/silty sand/sandy silt till deposit in borehole 101. The reddish brown layer was in a moist condition and contained traces of sand, gravel and shale fragments. The clayey silt till layer extended to the full depth of borehole exploration.

It should be noted that the silt/clay/sand/till soil is unsorted deposit; therefore, boulders and cobbles are anticipated.

Groundwater: Upon completion of drilling all monitoring wells by Shad and Associates Inc. were dry.

On March 9, 2018, ground water levels were measured at depths ranging from 2.8 to 4.2 m in boreholes 1, 3 to 5, 9 to 10 and 12. On March 16, 2018, groundwater levels were measured at depths ranging from 2.9 to 6.4 m in boreholes 1, 3 to 5, 8 to 10 and 12.

On January 6, 2023, groundwater levels were measured at depths ranging from 0.74 to 3.76 m in boreholes 1, 3 to 5, 9 to 10 and 12. The results are summarized on the Record of Borehole Sheets in Appendix C and Table 1.

4.2 GROUNDWATER LEVEL MONITORING

All current and past groundwater monitoring data is presented in Table 1. It should be noted that groundwater levels are subject to seasonal fluctuations. All groundwater levels were measured manually using an electric water level meter and with respect to the geodetic borehole elevations within the property boundary. The monitoring wells must be decommissioned, prior to construction, in accordance with Regulation 903 by a qualified contractor.

The interpreted groundwater flow direction is based on the 2018 and 2022 – 2023 round of water table elevation measurements, since this event provided the water table elevations from the majority of the monitoring wells. The interpreted local direction of hydraulic movement across the Site is inferred to be in a southern direction, towards the West Tributary of the Sixteen Mile Creek.

4.3 GROUNDWATER QUALITY

The groundwater samples collected from BH 1 and 10 in December 2022 were analyzed for the Municipality of Halton Sewers By-Law criteria. The results of chemical analysis (Table 2) indicate that the sample complies with the *Table 1 Limits for Sanitary & Combined Sewers Discharge* and *Table 2 Limits for Storm Sewer Discharge* for all parameters analyzed.

4.4 GROUNDWATER DISCHARGE ASSESSMENT

Presently, the groundwater onsite can be discharged to the Municipal sanitary/combined sewer system or storm sewer system with no additional filtration/treatment.

5.0 REVIEW AND EVALUATION

5.1 TEMPORARY DEWATERING ASSESSMENT

The excavation for the proposed two level underground parking structure will extend into native sandy silt soils. In order to protect the sides/bottom of the excavation from being disturbed by excess groundwater pressure, i.e., to prevent quicksand/dilating silt conditions, the groundwater table must be lowered 1.0 m below the bottom of the footing excavations.

Positive dewatering such as well points/eductors will be required for the proposed excavation. Onsite soil might be subject to localized piping during dewatering. Creation of piping channels may result in a substantial increase in the volume of both temporary dewatering and permanent drainage.

For the proposed two underground levels, groundwater is required to be drawn down 1 m below the underside of the combined footings. The assumed elevation of the footings is at approximately 178.20 masl for the North Block and 179.55 for the South Block. Therefore, groundwater will need to be lowered to an elevation of 177.20 for the North Block and 178.55 masl for the South Block.

The average ground water level recorded in the monitoring wells is at an elevation of 182.54 masl (Table 3), representing an approximate 3 – 5 m hydrostatic head requiring dewatering. The size of the shoring plan layout was assumed to cover approximately 165 m by 80 m and 210 m by 82 m for the North and South Blocks, respectively.

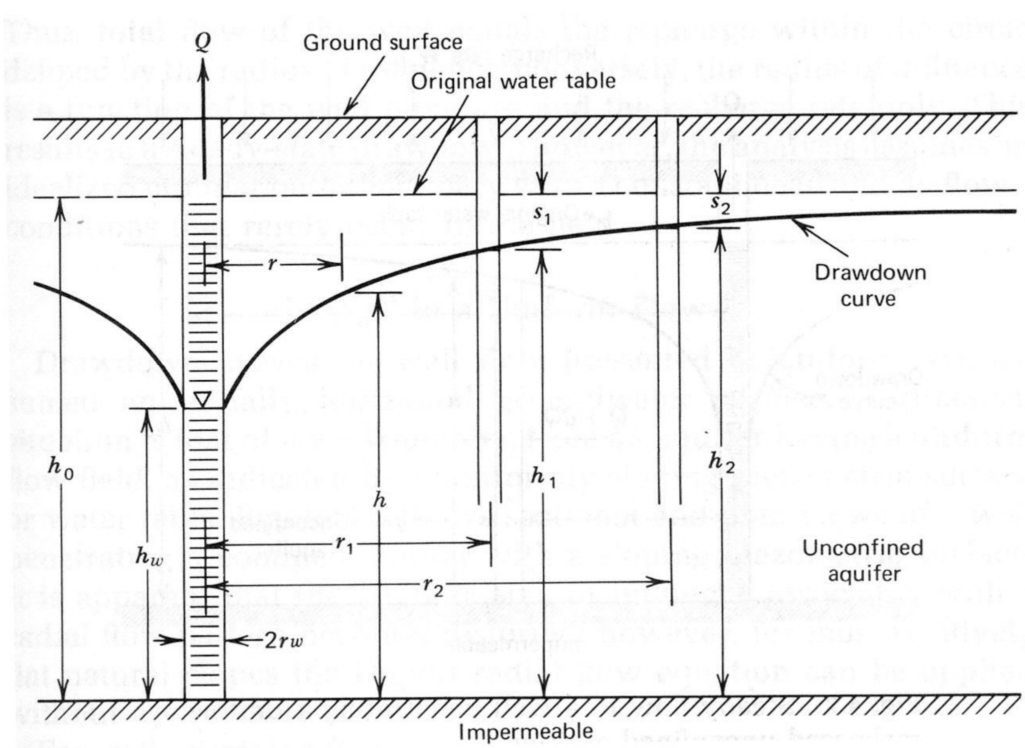
Theoretically, the groundwater drawdown for a single well pumping can be described as:

$$Q = -2\pi r K h \frac{dh}{dr} \tag{1}$$

And further we have:

$$h^2 = -\frac{Q}{\pi K}\ln(r / r_w) + h_w^{\ 2} \tag{2}$$

Where:
h [m] is the height of the water table above an impervious base
*Q [m^3/day]*is the rate of pumping discharge
K [m/day] is hydraulic conductivity
R [m] is the radius from the centre of well location
*r_w[m]*is the radius of pumping well (see Schematic A below).



Schematic A: Radial flow to an unconfined aquifer (Todd 1980)

5.1.1 Numerical Analysis

The abovementioned Site parameters were used to calculate the estimated steady state discharge rate for temporary construction dewatering. Groundwater monitoring data is presented in Table 3. The calculations for temporary dewatering rates are shown in Tables 4.

From the observed soil types and based on soil sample descriptions (*Todd, 1980; Mays, 2001; and Craig, 2004*), the average hydraulic conductivity (K) of the aquifer was estimated at 0.40 m/day.

The estimated steady state discharge rate for temporary construction dewatering was calculated at approximately:

Block	Discharge (m^3/day)
North	221
South	201

It should be noted that the initial drawdown pumping rate and accumulation from rainfall will likely be higher.

5.2 PERMANENT FOUNDATION DRAIN FLOW RATES

For the proposed redevelopment, it is understood that average ground floor slab elevation (FFE) is expected to range from elevations of 1847.65 to 188.0 meters above sea level (masl). The P2 floor slab elevation is expected to range from elevations of 179.70 to 181.05 masl.

A sub-floor Private Water Drainage System (PWDS) with perimeter/underfloor weeping tile is proposed below the P2 level slab. The invert of the PWDS is assumed to be approximately 0.5 m below the FFE of the P2 slab, i.e., at approximately 179.20 to 180.55 masl.

The proposed PWDS is shown in Drawing No. 6. The slotted pipes should slope to a sump at a minimum 1% slope. Perimeter drainage pipes, with a positive gravity outlet, should be solid PVC with a minimum of 0.5% slope. In addition, silt traps must be provided at convenient/accessible locations.

5.2.1 Numerical Analysis

The abovementioned Site parameters were used to calculate the estimated steady state discharge rate for the PWDS. Groundwater monitoring data is presented in Table 3. The calculations for permanent drainage flow rates are shown in Table 5.

From the observed soil types and based on soil sample descriptions (*Todd,* *1980; Mays, 2001; and Craig, 2004*), the average hydraulic conductivity (K) of the aquifer was estimated at 0.40 m/day.

The estimated steady state discharge rate for the PWDS was calculated at:

Block	Discharge (m^3/day)
North	117
South	94

5.3 MECP PERMIT TO TAKE WATER REQUIREMENT

The Permit to Take Water (PTTW) requirements for construction site dewatering have been updated to the current O.Reg.63/16 amendment to Environmental Protection Act. In accordance with the updated regulation, construction site dewatering will require a complete PTTW application when water takings greater than 400,000 L/day are predicted. Groundwater taking between 50,000 L/day and 400,000 L/day will require a PTTW through a limited online application process. Groundwater taking from a proposed building structure by means of a PWDS will require a PTTW when water taking is greater than 50,000 L/day. The complete permit application process for PTTW takes approximately twelve weeks to review and is required prior to applying for the discharge permits.

The anticipated steady state temporary dewatering discharge rate was calculated at 221 m^3/day and 201 m^3/day for the North and South Blocks, respectively. Therefore, a limited PTTW application will be required to be applied for with the MECP for each Block.

The steady state flow rate from the PWDS was calculated at 117 m^3/day and 94 m^3/day for the North and South Blocks, respectively. Therefore, a complete PTTW application for the PWDS will be required for each Block.

In accordance with the current Ontario Regulation 387/04 for Water Taking, every person to whom a permit has been issued under Section 34 of the Act shall collect and record data on the volume of water taken daily. The data collected shall be measured by a flow meter or calculated using a method acceptable to a Director.

5.4 Municipal Discharge Permit Requirements

The Municipality of Halton requires that any private water to be discharged into the city sewer system must have a permit or agreement in place in order to discharge; this applies to all water not purchased from the city water supply. For temporary dewatering during the construction phase, this includes all groundwater and storm water that is collected or encountered during site excavation. For the PWDS, this includes all groundwater that is constantly pumped as a result of the PWDS elevation located below the groundwater table elevation or through storm water infiltration.

The groundwater quality sample collected in December 2022 indicates that the water onsite could be discharged into the Municipal sanitary and combined sewer system or storm sewer system with no additional filtration or treatment. A short-term temporary discharge permit must be applied for construction dewatering with Municipality.

A long-term permanent discharge permit must be applied for the proposed PWDS since the drainage system is located below the long-term groundwater elevation. The permanent discharge permit will involve coordination with the mechanical and site servicing consultant to provide calculations and drawing specifications for the ultimate discharge location and the sampling port required by the Municipality.

5.5 Environmental Protection

The Site is located in the Sixteen Mile Creek drainage basin and a branch is approximately 30 m west of the Site. The Site is located within the Regional Municipality of Halton and there are potential potable groundwater issuers in the Vicinity of the Site. Therefore, the Site is located in a potable groundwater region as defined in Sections 35 to 37 of O.Reg. 153/04.

The proposed redevelopment plan will remove all the overburden to a depth of approximately 9 – 10 mbgs, from the interior Site area. Temporary groundwater dewatering will lower the groundwater table to below the underground parking foundation levels. The extracted water will be discharged into the sanitary sewer or into the storm sewer. Updated groundwater monitoring will be conducted by the dewatering contractor prior to and during construction activities to ensure that no additional adverse groundwater impacts are identified throughout the project’s construction.

6.0 CONCLUSIONS AND RECOMMENDATIONS

MCR Engineers Ltd. was retained to conduct a Geohydrology Assessment for the Site in relation to an administrative Plan of Subdivision and rezoning application. Etheridge Avenue bisects the Site, running west to east. The Site is presently a vacant lot.

The Site is proposed for residential development (Appendix B) and will consist of:

- **North Block:** A fifteen [15] storey building (Building 5), a twelve [12] storey building (Building 6), and a fourteen [14] storey building (Building 7) over two [2] levels of underground parking.
- **South Block:** A fifteen [15] storey building (Building 1), a thirteen [13] storey building (Building 2), an eleven [11] storey building (Building 3), and a fourteen [14] storey building (Building 4) over two [2] levels of underground parking.

The design grades are not known at this stage. Therefore, our recommendations should be considered preliminary. Our suggestions might be revised when the proposed Site/Foundation Plan becomes available.

However, the assumed finished floor elevation (FFE) at ground level is expected to be at an elevation of 187.65 to 188.0 meters above sea level (masl). The P2 FFE will be at an approximate elevation of 179.70 to 180.50 masl for the North Block and at 181.05 masl for the South Block.

Presently it is assumed that the proposed buildings will be supported by conventional spread/strip footings founded in silty to sandy/clayey silt soils. The size of the shoring play layout was assumed to cover approximately:

- **North Block:** 165 m by 80 m
- **South Block:** 210 m by 82 m

A sub-floor Private Water Drainage System (PWDS) with perimeter weeping tile will be required for the proposed development. A soldier pile and lagging shoring system is expected for temporary excavation.

The excavation for the proposed two level underground parking structure will extend into native sandy silt soils. In order to protect the sides/bottom of the excavation from being disturbed by excess groundwater pressure, i.e., to prevent quicksand/dilating silt conditions, the groundwater table must be lowered 1.0 m below the bottom of the footing excavations.

Positive dewatering such as well points/eductors will be required for the proposed excavation. Onsite soil might be subject to localized piping during dewatering. Creation of piping channels may result in a substantial increase in the volume of both temporary dewatering and permanent drainage.

For the proposed two underground levels, groundwater is required to be drawn down 1 m below the underside of the combined footings. The assumed elevation of the footings is at approximately 178.20 masl for the North Block and 179.55 for the South Block. Therefore, groundwater will need to be lowered to an elevation of 177.20 for the North Block and 178.55 masl for the South Block.

The average ground water level recorded in the monitoring wells is at an elevation of 182.54 masl (Table 3), representing an approximate 3 – 5 m hydrostatic head requiring dewatering.

The steady-state discharge rate for temporary construction dewatering was calculated at 221 m^3/day (40 USG/min) and 201 m^3/day (37 USG/min) for the North and South Blocks, respectively Therefore, based on the amended O.Reg. 63/16 to the Environmental Protection Act, a limited PTTW application will be required from the MECP, and a temporary discharge permit will be required from the MECP for each Block. It should be noted that the initial drawdown pumping rate and accumulation from rainfall will be higher and this should be confirmed by the dewatering contractor.

The steady state discharge rate for the PWDS was calculated at approximately 117 m^3/day (21 USG/min) and 94 m^3/day (17 USG/min) for the North and South Blocks, respectively. Therefore, a complete PTTW will be required from the MECP for the PWDS for each Phase. A long-term permanent discharge permit will be required from the Municipality since the drainage will be installed below the long-term groundwater elevation.

The selected dewatering contract must be performance driven and the contractor must provide a performance bond. In addition, upon completion of system's installation, the contractor must produce a written statement that "The system installed is robust enough to lower and maintain groundwater at least 1.0 m below the lowest footing elevation, without impacting the integrity of shoring or foundation soils."

The Zone of Influence (ZOI) for construction dewatering ranges from 26 to 35 m. The ZOI for permanent drainage ranges from 13 to 22 m. As the ZOI for construction dewatering and permanent drainage may intercept the branch of the Sixteen Mile Creek to the west and south, an infiltration gallery, **with approval from the Municipality and the MECP with an Environmental Compliance Approval (ECA),** could be implemented to offset the potential of drying out the creek.

Presently, the groundwater onsite can be discharged to the Municipal sanitary/combined sewer system or storm sewer system with no additional filtration/treatment.

The application process, where a PTTW is required, can take at least three months for a review by the MECP and is required to be approved prior to applying for discharge permits. It is recommended that applications to the Municipality for discharge permits be applied for at least three months prior to the required start dates. Applications are to be supported by drawings and calculations provided by the mechanical and the site servicing consultant and coordination is required amongst all disciplines.

7.0 REFERENCES

1. Ontario Ministry of the Environment. *Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act.* April15, 2011.
2. Ministry of Northern Development and Mines. *Quaternary Geology of Toronto and Southern Ontario - Southern, Sheet Map 2504,* 1980.
3. Ministry of Northern Development and Mines. *Bedrock Geology of Ontario-Southern Sheet,* 1991.
4. D.K. Todd, *Groundwater Hydrology,* 2nd Edition, John Wiley & Sons, New York, 1980.
5. L.W. Mays, *Water Resources Engineering*, 1st Edition, John Wiley & Sons, New York, 2001.
6. R.F. Craig, *Soil Mechanics*, 7th Edition, Spon Press, London, 2004.
7. Shad & Associates Inc. report titled, *Geotechnical Investigation Report, Proposed Residential Condominium Development, Framgard Property – Major Node, Regional Road 25, North of Britannia Road, Milton, Ontario,* prepared for Mattamy Willmott Limited, dated March 2018.
8. MCR report titled, *Geotechnical Report, Residential Development, Regional Road 25 and Britannia Road, Milton, Ontario,* prepared for Mattamy (Milton West) Limited, dated January 2024.

8.0 STATEMENT OF LIMITATIONS

MCR Engineers Ltd. (MCR) conducted the work associated with this report in accordance with the scope of services, time and budget limitations imposed for this work. The work has been conducted according to reasonable and generally accepted local standards for an environmental consultant at the time of the work. No other warranty or representation, expressed or implied, is included or intended in this report.

The work was designed to provide an overall assessment of the environmental conditions at the Site. The conclusions presented in this report are based on the information obtained during the investigation. The work is intended to reduce the client's risk with respect to environmental impairment. No work can completely eliminate the possibility of further environmental impairment on the Site.

It should be noted that subsurface conditions might vary at locations and depths other than those locations where borings, surveys or explorations were made by MCR. Other contaminants, not tested for in this work, may also potentially be present on the Site. Even with exhaustive investigation, it is not possible to warranty the Site will be free of contaminants. Should conditions, not observed during the work, become apparent, MCR should be immediately notified to assess the situation and conduct additional work, where required. The findings of this report are based on conditions as they were observed at the time of the work.

No assurance is made regarding changes in conditions subsequent to the time of the work. Remediation cost estimates is based on the available information. The estimated costs for remediation only represent the costs for the clean-up of known contaminants that have been identified during the work. Additional costs may be incurred as a result of other contaminants or areas of contamination identified by subsequent work.

Regulatory statutes are subject to interpretation. These statutes and their interpretation may change over time, thus these issues should be reviewed with appropriate legal counsel.

MCR relied on information provided by others in this report. MCR cannot guarantee the accuracy, completeness and reliability of the information provided by others, although MCR staff attempted to seek clarification on information provided and verifies authenticity, where practical.

The report and its attachments were prepared for and made available for the sole use of the client. MCR will not be responsible for any use or interpretation of the information contained in this report by any other party without the prior expressed written consent of MCR.

9.0 CLOSURE

In accordance with your request and authorization, MCR Engineers Ltd. completed this Geohydrology Assessment Report. This report presented the methodology, findings and conclusions of the investigation. The Statement of Limitations for all work performed as part of this investigation is included.

We trust that the information provided in this report is sufficient for your present requirements. Should you have any further questions, please do not hesitate to contact our office. Thank you for retaining MCR Engineers Ltd. for this project.

Respectfully,
MCR Engineers Ltd.

LICENSED PROFESSIONAL ENGINEER
R. J. SUKHU
100505663
PROVINCE OF ONTARIO

Prepared By:
Richard Sukhu, P.Eng., B.Eng.

Reviewed By:
Lad Rak, P.Eng., M.Eng., QP_{ESA}

Date of Issue: January 12, 2024

C-3 Excerpts from Water Balance Report (Jan 2024)

MCR ENGINEERS LTD.
GEO-ENVIRONMENTAL CONSULTANTS

111 ZENWAY BLVD., UNIT 4, VAUGHAN, ONTARIO, L4H 3H9
TEL: 416.675.0160 FAX: 905.851.1722
office@mccrak.com

1271 DENISON ST., UNIT 45, MARKHAM, ONTARIO, L3R 4B5
TEL: 905.470.0160 FAX: 905.475.6371
denison@mccrak.com

WWW.MCCRAK.COM

G 5820 JANUARY 2024

REPORT
WATER BALANCE ASSESSMENT
NORTHWEST CORNER OF REGIONAL ROAD 25
AND BRITANNIA ROAD
MILTON, ONTARIO

DISTRIBUTION:

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MATTAMY HOMES CANADA
7880 KEELE STREET
VAUGHAN, ONTARIO
L4K 4G7

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FIGURES

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APPENDICES

1.0 INTRODUCTION

MCR was retained by Mattamy Homes Canada (the Client) to carry out a water balance assessment for the proposed residential development, located at North-western corner of the intersection of Regional Road 25 and Britannia Road, Milton, Ontario.

The water balance assessment includes an evaluation of precipitation, potential evapotranspiration, surface runoff and infiltration conditions of the proposed residential development. The method developed by Thornthwaite and Mather, was applied in the assessment. Results will be used for storm water management planning of the proposed development.

1.1 SCOPE OF WORK

The objectives of the Water Balance Assessment are to determine the following:

- Hydrogeological conditions of the Site, including the stormwater, surface run off, subsurface flow patterns and their contribution to water balance under preconstruction and postconstruction conditions.
- Review the available background information for the Site obtained from MCR's files and architectural drawings.
- Using Thornthwaite and Mather water balance method provide preliminary water budget analysis (i.e., surface ET, surface runoff, infiltration to soil) for pre- and post-development, to mitigate impacts of increased runoff and further to manage runoff as close to its source as possible.
- Summarize the findings in a Water Balance Assessment Report.

1.2 SITE DESCRIPTION

The Site is located at the northwestern corner of Regional Road 25 and Britannia Road, in a mixed-use rural, residential, and commercial area of the city of Milton, Ontario. The Site is irregular in shape with an approximate area of 41,511 m^2.

The Site is bounded by a pond to the north, Regional Road 25 to the east, Britannia Road to the south, and a pond/channel to the west. Etheridge Avenue bisects the Site, running west to east. The Site is presently a vacant lot. The Site is legally described as: Part of Lot 6, Concession 2, Municipality of Milton Ontario. The topographic surveys are attached in Appendix A.

1.3 PROPOSED DEVELOPMENT

The Site is proposed for residential development (Appendix B) and will consist of:

- North Block: A thirteen [13] storey building (Building 5B) and an eleven [11] storey building (Building 5B) over three [3] levels of combined underground parking.
- Central Block: A fifteen [15] storey building (Building 4) over three [3] levels of underground parking.
- South Block: A thirteen [13] storey building (Building 1A), a fifteen [15] storey building (Building 1B), a fifteen [15] storey building (Building 2), a thirteen [13] storey building (Building 3A) and an eleven [11] storey building (Building 3B) over three [3] levels of underground parking.

The finished floor elevation (FFE) at ground level is expected to be at an elevation of 186.0 meters above sea level (masl) for the North and Central Blocks and 184.0 masl for the South Block.

1.4 PROPERTY OWNERSHIP

The Site is owned and intended for redevelopment by Mattamy Homes Canada. The Owner is represented by Ms. Christine Chea, with the following contact information:

Ms. Christine Chea, MCIP, RPP
Direction, Development, GTA Urban
7880 Keele Street
Vaughan, Ontario
L4K 4G7
Email: christine.chea@mattamycorp.com

1.5 REVIEW OF PREVIOUS REPORTS

The following geo-environmental reports and drawings were provided for review prior to initiating the water balance assessment:

MCR report titled, *Preliminary Geotechnical Report, Residential Development, Northwestern Corner of Regional Road 25, and Britannia Road, Milton, Ontario*, prepared for Mattamy Homes Canada, dated January 2023.

MCR report titled, *Preliminary Geohydrology Assessment, Residential Development, Northwestern Corner of Regional Road 25, and Britannia Road, Milton, Ontario*, prepared for Mattamy Homes Canada, dated January 2023.

Shad & Associates Inc. report titled, *Geotechnical Investigation Report, Proposed Residential Condominium Development, Framgard Property – Major Node, Regional Road 25, North of Britannia Road, Milton, Ontario*, prepared for Mattamy Willmott Limited, dated March 2018.

Survey Drawing Plan of Topography and Sketch showing Elevations of Part of Lot 5, Concession 2, New Survey and Part of Plan 20M-1165 Town of Milton, Regional municipality of Halton, prepared by Rady-Pentek & Edward Surveying Ltd. And dated February 5th, 2018.

Architect Drawings Framgard Mattamy, prepared by Core Architects, and dated January 9, 2023 .

2.0 HYDROGEOLOGICAL CONDITIONS

2.1 PHYSICAL SETTING

The Site is located in the Town of Milton and is situated in a mixed-use rural, residential, and commercial area. The nearest major intersection is Regional Road 25 and Britannia Road, located southeast of the Site. A branch of The West Tributary of the Sixteen Mile Creek is located approximately 30 m west of the Site.

The Site is located at an elevation of approximately 184 to 186 m above sea level (asl) and the topography across the Site slopes from the north to south. Surrounding area slopes from northwest to southeast, towards the Sixteen Mile Creek.

The Site is bounded by the following properties/features:

North	A pond
South	Britannia Road
East	Regional Road 25
West	Pond / Channel

2.2 TOPOGRAPHY

According to the topographic map, published by the Government of Canada; Natural Resources Canada at the Government of Canada website: http://atlas.gc.ca/toporama/en/index.html, the ground surface at the Site slopes from north to south and the surrounding area sloping from northeast to south west towards the Sixteen Mile Creek.

2.3 REGIONAL GEOLOGY AND HYDROGEOLOGY

According to the geological map entitled "Quaternary Geology of Ontario, Southern Sheet”, published by the Ontario Ministry of Development and Mines, dated 1991, the overburden in the study area consists mainly of Halton till, predominantly silt and clay, minor sand, basin, and quiet water deposits. Groundwater flow is expected to be directed southwest towards the Sixteen Mile Creek.

According to the Ontario Ministry of Development and Mines, Map No. 2554 "Bedrock Geology of Ontario, Southern Sheet, 1991", the bedrock typically consists of Upper Ordovician shale, limestone, dolostone and siltstone Queenston Formation. On a regional scale, groundwater is expected to flow south-west, towards the Sixteen Mile Creek.

2.4 LOCAL GEOLOGY AND HYDROGEOLOGY

On a local scale, geological conditions and hydrogeology are similar to the ones at a regional scale. Locally, near surface groundwater flow may be influenced by underground structures (e.g., service trenches, catch basins, and building foundations or surface watercourses). No surface water features are present onsite and there are no Provincially Significant Wetlands in the vicinity of the Site.

Background review and field investigations identified that a Natural Heritage System (NHS), a branch of Sixteen Mile Creek, traversing the west side of the subject property, in a northwest-to-southeast orientation. Elevations of the creek bad varied from 182.5 to 181.5 masl, from upstream to downstream.

3.0 REVIEW OF SITE INVESTIGATIONS

3.1 OVERVIEW OF SITE INVESTIGATION

Initially, twelve boreholes (BH 1 to BH 12) were drilled by Shad & Associates Inc. from February to March 2018 to depths ranging from 7.80 to 8.10 m. Boreholes 1, 3 to 5, 8 to 10 and 12 were equipped with monitoring wells for long-term groundwater monitoring and sampling.

Nine boreholes (BH 101 to BH 109) were drilled by MCR in December 2022 to January 2023 to depths ranging from 7.30 to 21.40 m.

The borehole locations are shown in Drawing No. 1 and the records are presented in Appendix C.

3.2 MONITORING WELL INSTALLATION

It is assumed that all monitoring wells by Shad and Associates Inc. were installed with a 50 mm diameter schedule 40 PVC pipe and a 3.05 m long slotted well screen. Well screens were surrounded by a silica sand pack to at least 0.6 m above the top of screen with a bentonite seal extending from above the sand pack to within 0.5 m of the ground surface. All monitoring wells were completed with a flush mounted cover at ground surface. Monitoring well installation was done in accordance with the Ontario Water Resources Act, Sections 35 to 50.

3.3 ELEVATION SURVEYING

MCR elevations referred to in this report are metric and geodetic and are interpolated from the provided topographic survey prepared by Rady-Pentek & Edward Surveying Ltd., dated February 9 and April 13, 2018. Borehole elevations are shown on the borehole logs in Appendix C.

3.4 GROUNDWATER LEVEL MONITORING

Groundwater levels were recorded from all available monitoring wells over various dates and the data is presented in the enclosed TABLES 1 and 2. Water levels were measured manually with an electric water level meter. Water levels were measured with respect to geodetic borehole elevations within the site boundaries.

4.0 INVESTIGATION RESULTS

4.1 GEOLOGY

The ground surface elevation across the Site varies from 187.50 masl (BH 104) to 184.70 masl (BH 1). Based on the investigations by MCR and Shad and Associates Inc., the geologic formations beneath the Site are illustrated in borehole logs (Appendix C) and include the following (from surface to depth):

Please note that boreholes 102, 103, 106 and 108 were straight drilled to 9.15 m due to proximity to Shad and Associates Inc. boreholes.

Fill: Compact fill material was encountered at the surface of all boreholes. The fill material extended to depths ranging from 0.4 to 0.9 m. The fill consisted of silty sand/sandy silt/clayey silt/silty clay, sand, and gravel soils. The brown/dark brown to reddish brown fill was in a moist condition and contained some to trace of organics, clay, gravel, and rootlets.

Silty Sand/Sandy Silt: A dense silty sand/sandy silt till layer was encountered below the fill in boreholes 104, 105, 107 and 109. The brown silty sand/sandy silt layer was in a moist condition and contained traces of clay. The silty sand/sandy silt layer extended to the full depth of borehole 104 and a depth of 2.30 m in boreholes 105, 107 and 109.

Clayey Silt/Silty Clay (Till): A very stiff to hard clayey silt/silty clay till layer was encountered below the fill and silty sand/sandy silt layer in all boreholes (except 102, 103, 106 and 108). The reddish brown to grey clayey silt/silty clay till layer was in a moist to wet condition and contained some to trace of sand, gravel, and shale fragments. The clayey silt/silty clay till layer extended to the full depth of boreholes 2, 3, 5, 8, 11 and 109 and to depths ranging from 4.55 to 10.65 m in all other boreholes.

Sand and Gravel/Silty Sand/Sandy Silt (Till): A very dense sand and gravel/silty sand/sandy silt till deposit was observed below the clayey silt/silty clay till layer in all boreholes. The brown to reddish brown sand and gravel/silty sand/sandy silt (till) deposit was in a moist to wet condition and contained traces of clay, gravel, and shale fragments. The sand and gravel/silty sand/sandy silt till layer extended to a depth of 18.30 m in borehole 101 and to the full depth of all other boreholes.

Clayey Silt Till: A hard layer of clayey silt till was detected below the sand and gravel/silty sand/sandy silt till deposit in borehole 101. The reddish-brown layer was in a moist condition and contained traces of sand, gravel, and shale fragments. The clayey silt till layer extended to the full depth of borehole exploration.

4.2 GROUNDWATER LEVELS

The water levels in the on-site wells were used to evaluate both the groundwater flow direction and the depth to water table. Groundwater level in monitoring wells varied from 5.30 m (BH2) to 12.20 m (BH101) in depth, with 7.45 m on average, measured from August 10/2018 to April 15/2019. All groundwater measurement data is presented in the enclosed Table 1.

Table 1. Groundwater Level Monitoring Results

MONITORING WELL ID	GROUND SURACE ELEVATION (masl)	GROUNDWATER		DATE OF MEASUREMENT (mm/dd/yyyy)
		WATER LEVEL (mbgs)	ELEVATION (masl)	
BH 1	184.70	2.80	181.90	03/09/2018
		2.90	181.80	03/16/2018
		2.80	181.90	01/06/2018
BH 3	185.80	3.70	182.10	03/09/2018
		3.60	182.20	03/16/2018
		3.74	182.06	01/06/2018
BH 4	185.10	3.60	181.50	03/09/2018
		3.50	181.60	03/16/2018
		3.26	181.84	01/06/2018
BH 5	186.60	4.20	182.40	03/09/2018
		4.30	182.30	03/16/2018
		0.74	185.86	01/06/2018
BH 8	186.70	*DRY*	-	03/09/2018
		6.40	180.30	03/16/2018
		NOT FOUND	-	01/06/2018
BH 9	186.70	2.90	183.80	03/09/2018
		2.90	183.80	03/16/2018
		3.76	182.94	01/06/2018
BH 10	186.60	2.90	183.70	03/09/2018
		3.00	183.60	03/16/2018
		2.94	183.66	01/06/2018
BH 12	186.60	3.60	183.00	03/09/2018
		3.60	183.00	03/16/2018
		3.72	182.88	01/06/2018
Min	184.70	0.74	180.30	-
Max	186.70	6.40	185.86	-
Average	186.10	3.43	182.68	-

5.0 WATER BALANCE ASSESSMENT

A water balance is to determine the amount of surplus water potentially generated, and the quantity of infiltration change due to the increase in impermeable surface of the proposed development. The data was then used in the evaluation of options to manage the surplus.

Thornthwaite and Mather method was applied in the calculations of potential evapotranspiration, soil moisture storage/retention, and total water surplus. Water surplus was calculated as a final product of the total water available in each period to run off, as a surface overland flow, and/or infiltrate to the ground and recharge the groundwater table.

The water balance assessment was prepared according to the "Hydrogeological Assessment Submissions: Conservation Authority Guidelines to Support Development Application (2013)

5.1 CLIMATE DATA & SOIL PARAMETERS

Climate data used in water balance calculation was the summarized monthly average of daily temperature and precipitation for the period of 1981 to 2010, obtained from Environment Canada's Oakville Southeast WPCP (Climate ID: 615N745). The weather station is about 10 km away from the Site and located in the east of the City of Milton, Ontario.

The calculated daily average temperature is about 8.0 ^{o}C of 30 year's average. Yearly total of average precipitation is 806.7 mm, consisting of 725.6 mm rainfall and 81.0 mm of snow melt water. Detailed variation of climate data in monthly average is shown in Figure 1.

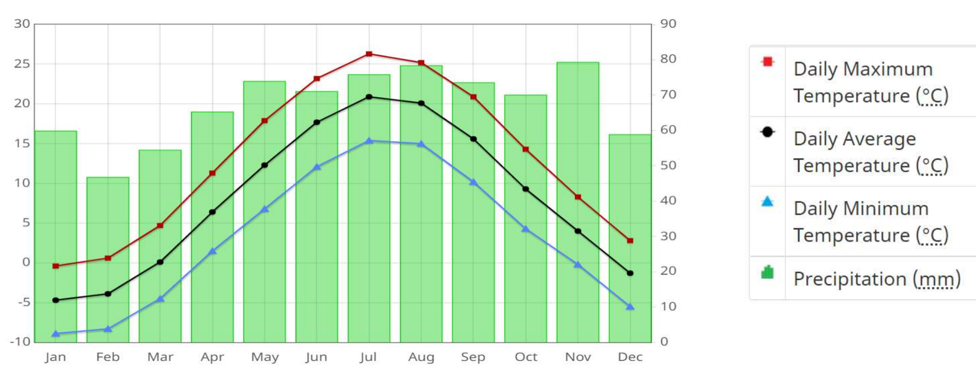


Figure 1. Canadian Climate Normal, 1981 to 2010, Oakville Southeast WPCP

The soils underlying the Site are described as Class C, very loose to compact silty sand/sand in fill material, and covered with some silt loams, with a low runoff potential and moderated infiltration. The water holding capacity was determined from tables provided in the Ontario's Stormwater Management Planning and Design Manual (MOE, 2003b), which relate water holding capacity to soil type and land use.

5.2 THORNTHWAITE AND MATHER MODEL

An accounting type procedure was utilized, within Thornthwaite and Mather Model, to analyze the allocation of water among various components of a hydrologic cycle.

Inputs to the model were monthly temperature, precipitation, and the site latitude. Outputs include monthly potential and actual evapotranspiration, infiltration, soil moisture storage change, surplus, and runoff. The annual water balance can be expressed as

$$P = ET + R + I + ST$$

Where	P	is monthly averages of precipitation
	ET	is evapotranspiration,
	R	is surface water runoff.
	I	is infiltration, and
	ST	is soil moisture storage change.

Precipitation (P)

Based on the 30-year average (1981-2010) for the Environment Canada's Weather Station, Oakville Southeast WPCP, Ontario (Climate ID: 615N745), the average precipitation is 806.7 mm/year. The monthly precipitation distribution is presented in Table 2.

Table 2. Monthly Average of Temperature and Precipitation

	Temperature (^{o}C)	Precipitation (mm)	Rainfall (mm)	Snowmelt (cm)
January	-4.7	59.8	31.5	28.3
February	-3.9	46.8	30.7	16.1
March	0.1	54.4	37.2	17.2
April	6.4	65.2	63.1	2.1
May	12.3	73.9	73.9	0.0
June	17.7	71.0	71.0	0.0
July	20.9	75.8	75.8	0.0
August	20.1	78.3	78.3	0.0
September	15.6	73.5	73.5	0.0
October	9.3	70.0	70.0	0.0
November	4.0	79.3	76.8	2.5
December	-1.3	58.8	43.9	14.9
Year	8.0	807	726	81

Note: *Data was obtained from Weather Station 615N745, Oakville Southeast WPCP*

Storage Change (ST)

It should be noted that for the topography, soil conditions (silty sand to sand) and vegetative cover (moderate to deep rooted crops) of the Site, the Long-term annual storage change is 0, although there may have been some variations on a monthly basis.

Evapotranspiration (ET)

Thornthwaite water balance model was applied in potential evapotranspiration (ET) calculation. ET is estimated from monthly temperature and is defined as a water loss from a homogeneous, vegetation covered area with a sufficient water resource. The method is based on an annual temperature efficiency index *I*, defined as the sum of 12 monthly values of heat index *I*. Each index *I* is a function of the mean monthly temperature T, in degrees Celsius, as follows:

$$I = \left(\frac{T}{5}\right)^{1.514}$$

Potential evapotranspiration is calculated by the following formula:

$$ET(0) = 1.6\left(\frac{10\,T}{I}\right)^{c}$$

where ET(0) is the potential evapotranspiration at 0° latitude in centimeters per month; and c is an exponent to be evaluated as follows:

$$c = 0.000000675\,I^3 - 0.0000771\,I^2 + 0.01792\,I + 0.49239$$

At the latitude other than 0° potential evapotranspiration is calculated by

$$ET = K\,ET(0)$$

in which K is a constant for each month, varying as a function of latitude, shown in Table 3. The latitude for the weather station 615N745, at Oakville Southeast WPCP is N-43°29'.

Table 3. Adjustment Factor for use in Thornthwaite Formula

Latitude	0	10	20	30	35	40	45	50
January	1.04	1.00	0.95	0.90	0.87	0.84	0.80	0.74
February	0.94	0.91	0.90	0.87	0.85	0.83	0.81	0.78
March	1.04	1.03	1.03	1.03	1.03	1.03	1.02	1.02
April	1.01	1.03	1.65	1.08	1.09	1.11	1.13	1.15
May	1.04	1.08	1.13	1.18	1.21	1.24	1.28	1.33
June	1.01	1.06	1.11	1.17	1.21	1.25	1.29	1.36
July	1.04	1.08	1.14	1.20	1.23	1.27	1.31	1.37
August	1.04	1.07	1.11	1.14	1.16	1.18	1.21	1.25
September	1.01	1.02	1.02	1.03	1.03	1.04	1.04	1.06
October	1.04	1.02	1.00	0.98	0.97	0.96	0.94	0.92
November	1.01	0.98	0.93	0.89	0.86	0.83	0.79	0.76
December	1.04	0.99	0.94	0.88	0.85	0.81	0.75	0.70

Water Surplus

It is a widespread practice and an acceptable method, by most Conservation Authorities, to provide estimates of water surplus using Thornthwaite and Mather approach. Water surplus has been calculated as the residual of precipitation minus evapotranspiration. Results of the calculations are presented in Table 4.

Table 4. Temperature, Precipitation, Evapotranspiration and Water Surplus

Month	Temperature	Precipitation	Rainfall	Snowmelt	Potential E.T.	Water Surplus
	(oC)	(mm)	(mm)	(cm)	(mm)	(mm)
January	-4.7	59.8	31.5	28.3	0	60
February	-3.9	46.8	30.7	16.1	0	47
March	0.1	54.4	37.2	17.2	0	54
April	6.4	65.2	63.1	2.1	32	33
May	12.3	73.9	73.9	0.0	73	1
June	17.7	71.0	71.0	0.0	110	-39
July	20.9	75.8	75.8	0.0	134	-59
August	20.1	78.3	78.3	0.0	120	-42
September	15.6	73.5	73.5	0.0	79	-5
October	9.3	70.0	70.0	0.0	41	29
November	4.0	79.3	76.8	2.5	14	66
December	-1.3	58.8	43.9	14.9	0	59
Year	8.0	807	726	81	603	204

There are two principal components of the calculated water surplus, infiltration, and surface runoff. Infiltration portion of the surplus is estimated by applying the infiltration factors provided in the Stormwater Management Planning and Design Manual of Ontario (MPDMO). The infiltration factors are determined by summing a factor for topography, soils, and land covers. The remaining portion of the water surplus will be considered as surface flow runoff.

Infiltration and Runoff

Soil infiltration refers to the ability of the soil to allow water to move into and through the soil profile. Infiltration allows the soil to temporarily store water, making it available for use by plants and soil organisms. Amount of infiltration is related with on-site soil type, topography, and surface cover. When rainfall is received, at a rate that exceeds the infiltration rate of a soil, runoff moves downslope or ponds on the surface in level areas.

Amount of infiltration is defined by total amount of water surplus, times an infiltration factor. Runoff is calculated as the residual of precipitation surcharge minus the amount of infiltration. Infiltration factor is determined by summing a factor for topography, soils, and vegetation cover. Table 5 shows the infiltration factors, applied in the water balance assessment, obtained from the provincial “Stormwater Management Planning and Design Manual (SMPDM)”, Section 53 of the Ontario Water Resources Act, dated March 2003:

Tables 5. Infiltration Factors from the SMPDM, Ontario Water Resources

	Characteristics	Factor
Topography	Flat Land, average slope < 0.6 m/km	0.3
	Rolling Land, average slope 2.8 to 3.8 m/km	0.2
	Lilly Land, average slope 28 to 47 m/km	0.1
Soils	Tight impervious clay	0.1
	Medium combinations of clay and loam	0.2
	Open sandy loam	0.4
Cover	Cultivated Land	0.1
	Woodland	0.2

5.3 LAND USE

Land use in water balance assessment, for the proposed development, was classified according to the Overall Site Plan Drawing A100, prepared by Core Architects, and dated January 9th, 2023, and the Topographic Survey Drawing, prepared by Rady-Pentek & Edward Surveying Ltd., dated February 9, 2018. Land use is classified as building coverage, road/driveway, parking/paved area and landscaped open space.

The water balance assessment was completed on a Site scale according to the overall site plan, Drawing A100 and statistics Drawing A001. Land use at pre-development is considered as agricultural land and/or landscaped area with a single farmer's residence (Table 6).

Table 6. Pre-Development Area Classification

Land Use	Percentage of Site (%)	Area (m^2)
Landscaped area / Permeable	92.3	38311
Permeable Parking/Paved Area	0.5	200
Road / Driveway / Parking	6.0	2500
Building Coverage	1.2	500
Total	100.0	41511

Building coverage, road/driveway, parking, and paved area are impervious. The total impervious area will be approximately 20601 m^2, at post development, and represents 49.7% of the total 41511 m^2 area (Table 7).

Table 7. Post Development Area Classification

Land Use	Percentage of Site (%)	Area (m^2)
Landscaped area / Permeable	55.0	22849
Permeable Parking/Paved Area	1.6	680
Road / Driveway / Parking	20.8	6818
Building Coverage	22.6	9364
Total	100.0	41511

5.4 WATER BALANCE ASSESSMENT

The meteorological data of yearly total, summarized from monthly averages of precipitation, evapotranspiration, infiltration, and runoff, were listed in Table 8. Detailed information of the collected climate data, soil classification and water balance calculations are enclosed in Appendix D.

Table 8. Summarized Climate data of yearly average.

Land Surface	Precipitation (mm)	Evapotrans-piration (mm)	Infiltration (mm)	Surface Runoff (mm)
Topsoil/pervious soil	807	603	71	133
Impervious soil	807	161	0.0	646

The Site's latitude, longitude, and an estimate of the water holding capacity of the soil was also an input to the model. The water holding capacity has been estimated based on soil and land use characteristics of the study area under Existing and Proposed conditions. Currently, at predevelopment stage, the area of proposed development consists of 98% pervious soils.

The soils on Site are covered by fine sand/sandy silt in fill material, described as Class C, Silt Loams with a low runoff potential and moderated infiltration. The water holding capacity was determined from tables provided in the Ontario's Stormwater Management Planning and Design Manual (MOE, 2003b), which relate water holding capacity to soil type and land use.

The infiltration factor was defined by summing a factor for topography, soils, and the characteristics of surface cover. Details for infiltration factor determination were listed in Table 9.

Table 9. Applied Infiltration Factors

Topography Classification	Factors
Topography, flat (aver slope less than 1.0 m/km)	0.15
Topsoil / Sandy Silt / Silt Loam	0.1
Land type - cultivated	0.1
Total	0.35

5.5 RESULTS AND DISCUSSIONS

The water balance calculations for pre and post development conditions are presented in Tables 10 and 11. Water surplus is the total water available, in a given time period to run off as surface overland flow, and/or to infiltrate to the ground and to recharge the groundwater table.

Based on the water balance calculations, it is estimated that there will be an increase in the amount of water surplus, from predevelopment conditions to the proposed post development conditions of approximately 7772 m^3 annually, calculated as $\delta = 1493 + 16073 - 2750 - 7043 = 7772$ m^3.

Table 10. Computation for predevelopment water balance

Land Use	Area (m^2)	Precipi-tation (m^3)	Evaptrans-piration (m^3)	Infiltration (m^3)	Runoff (m^3)
Landscaped area / Permeable	38011	30675	22921	2714	5040
Paved Drive Way / Road	500	404	302	0	66
Paved Public Parking	2500	2018	404	0	1614
Building Coverage	500	404	81	0	323
Total	41511	33499	23706	2714	7043

Table 11. Computation for post development water balance

Land Use	Area (m^2)	Precipi-tation (m^3)	Evaptrans-piration (m^3)	Infiltration (m^3)	Runoff (m^3)
Landscaped area / Permeable	21529	17374	12982	1537	2855
Paved Drive Way / Road	2000	1614	323	0	265
Paved Public Parking	8618	6955	1391	0	5564
Building Coverage	9364	7557	1511	0	6045
Total	41511	33499	16207	1537	14729

The reduction of infiltration of 1177 m^3 is determined by subtracting the post development infiltration of 1537 m^3 (Table 11) from the predevelopment condition's infiltration total of 2714 m^3 (Table 10) (2714 – 1537 = 1177 m^3). The reduction in the amount of infiltration is due to the increase in potential surface runoff, caused by the increase in impervious area and decrease in pervious surfaces for infiltration.

Infiltration targets can be achieved through the incorporation of a variety of stormwater management practices including reduced lot grading, roof leaders discharging to ponding/storage areas or soak away pits, infiltration trenches and grassed swales. In addition to addressing the increase in peak flow and volume, storm water management controls should concentrate on enhancing infiltration within the developed area to maintain the hydrological conditions of the site and nearby surface water features unchanged.

5.6 NATURAL HERITAGE SYSTEM

The Natural Heritage System (NHS), Sixteen Mile Creek, traverses the west side of the subject property, in a north-south orientation. MCR understands that the NHS will be kept with no change to its size, flow direction, area location, vegetation cover, soil composition and groundwater conditions during and post development. However, the quantities of water flow in and run out may vary and depend on the development compositions of its building area, surface flow conditions and the area of increased impervious pavement.

Results of water balance assessment indicate that comparing with predevelopment conditions, there would be about 7686 m^3 net increase, in surface water runoff into NHS yearly, under post development conditions. To keep the NHS at no change in size, vegetation cover and groundwater conditions, the 7686 m^3 extra surface runoff must be controlled by storm water management facilities.

6.0 MINTIGATION PLANS

6.1 INFILTRATION REDUCTION

The amount of potential Infiltration reduction was calculated as the difference of yearly total infiltration amount between predevelopment and post development. Potentially, this amount has been calculated as approximately 1177 m^3/year, assuming total precipitation has no change.

The calculated infiltration reduction is intended to provide an acceptable estimation based on current knowledge and evaluation of the conditions within the Site, and to assist with the design of proposed practical infiltration systems under post development conditions.

As a result, to keep base flow and/or groundwater regimes with no change for the post development conditions, an infiltration system is recommended. The estimated recharge to groundwater system would be about 1177 m^3 per year.

6.2 PROPOSED INFILTRATION GALLERY

Under the proposed development conditions, the amount of infiltration would decrease as of the increase in impervious area and decrease in permeable surfaces. Additional measures would need to be considered to promote evapotranspiration and infiltration on-site and to reduce runoff. As a result, a storm infiltration gallery is recommended. The calculated recharge to groundwater system would be about 1177 m^3 per year.

The size of the infiltration gallery can be calculated by the following equation:

$$A = 1000\ V/(P\ n\ \Delta t)$$

Where A is bottom area of the infiltration system (m^2)
V is the total volume of rainwater to be infiltrated.
P is the in situ tested infiltration rate (mm/hour)
n is the porosity of the storage media, $n = 0.4$ for clear stone
Δt is retention time.

The proposed infiltration gallery should rest on a bed of size 19 mm clear stone at least 200 mm thick. The stone bed would act as a barrier to prevent fine/suspension material influx into the infiltration gallery. Selected on site native excavated soils can be used as backfill after the infiltration gallery was constructed, provided that the excavated materials are not allowed to become wet. The excavated till will be lumpy and very sensitive to moisture content.

To eliminate the potential impact from seepage to the building foundations, the infiltration gallery must be kept at a minimum distance of 5 m away from the building envelope.

The infiltration gallery requires an approval from the Municipality and from the MECP/Environmental Compliance Approval (ECA),prior to installation.

6.3 SEDIMENT AND EROSION CONTROL

A sediment control plan will be required to protect the surface water directly flowing to the NHS, during construction/excavation in storm seasons. It is expected that berms and silt fence will be used to divert and control surface runoff from concentrated flows entering the NHS.

It is recommended that a visual inspection of the erosion and sediment control take place, by the site management, during the raining seasons to ensure that adequate sediment removal is taking place.

6.4 GROUNDWATER MONITORING FOR NHS

MCR understands that the NHS will be kept with no change to its size, flow direction, area location, vegetation cover, soil composition and groundwater conditions during and post development. Monitoring wells along the creek channel will be installed to monitor the groundwater conditions of the NHS.

Groundwater within the NHS system will be monitored from preconstruction to during and continued to post construction, with a frequency of minimum twice a month and will last for three years.

The data collected during the predevelopment phase will be used to calibrate and verify the existing conditions of water balance model. Data collected during construction and post construction will be compared with the data of preconstruction to re-evaluate the results.

6.5 CONTINGENCY PLAN

As described above, there will be a comprehensive program in place to monitor groundwater level and the potential impact on the features of the NHS during the proposed development construction. The need for mitigation will be triggered, during the dewatering, excavation, and substructures construction period, when sedimentation, erosion, groundwater level reduction or flooding would be observed within the receiving features of the NHS. Mitigation activities and contingency plans may include, but not be limited to, the following:

- Should water levels within the monitored NHS area decrease due to construction dewatering and/or increase during unusual storm events or become higher than expected to cause flooding or overland flow to surface water features, use of construction sediment control methods such as straw bales and silt fencing will be used to prevent turbid water from entering the NHS and the creek.

- Should water quality discharged, from construction dewatering, surpass the acceptable limit, sediment control will be reconstructed and the interaction between the excavation and surrounding water table will be limited.

- Should sediment or erosion, on top of slope surface or within the area of NHS located be observed, a reconstruction of sediment control will be carried out. Straw bales and silt fencing will be used to prevent the sediment or erosion.

7.0 CONCLUSIONS AND RECOMMENDATIONS

MCR was retained to conduct a Water Balance Assessment for the Site in relation to the proposed redevelopment. The Site is located at north-western corner of the intersection of Regional Road 25 and Britannia Road, in a mixed-use rural, residential/commercial area of the City of Milton, Ontario. The Site is irregular in shape, with an area of approximately 41511 m^2.

The Site is bounded by a pond to the north, Regional Road 25 to the east, Britannia Road to the south, and a pond/channel to the west. Etheridge Avenue bisects the Site, running west to east. The Site is presently a vacant lot and currently does not have a Legal description. The topographic surveys are attached in Appendix A.

Thornthwaite and Mather method was applied in the calculations of potential evapotranspiration, soil moisture storage/retention, and total water surplus. Water surplus was calculated as a final product of the total water available in each period to run off, as a surface overland flow, and/or infiltrate into the ground and to recharge the groundwater table.

The soils underlying the Site are described as Class C, Sandy Silt to silty clay in fill material, and covered with some silt loams, with a low runoff potential and moderated infiltration. The water holding capacity was determined from tables provided in the Ontario's Stormwater Management Planning and Design Manual (MOE, 2003b), which relate water holding capacity to soil type and land use.

Based on the water balance calculations, it is estimated that there will be an increase in the amount of water surplus, from predevelopment conditions to the post development conditions, of approximately 7499 m^3 annually.

The reduction of infiltration of 1177 m^3 is determined by subtracting the post development infiltration of 1537 m^3 (Table 11) from the predevelopment condition's infiltration total of 2714 m^3 (Table 10) (2714 – 1537 = 1177 m^3). The reduction in the amount of infiltration is due to the increase in potential surface runoff, caused by the increase in impervious area and decrease in pervious surfaces for infiltration.

Additional measures are considered to promote evapotranspiration and infiltration on-site and to reduce runoff. As a result, a storm infiltration gallery is recommended.

To eliminate the potential impact from seepage to the building foundations, the infiltration gallery must be located at a minimum distance of 5 m away from the building envelope.

It is reiterated that the infiltration gallery requires an approval from the Municipality and from the MECP/ Environmental Compliance Approval (ECA), prior to installation.

A sediment control plan will be required to protect the surface water directly flowing to the NHS, during construction/excavation in storm seasons. It is expected that berms and silt fence will be used to divert and control surface runoff from concentrated flows entering the NHS.

Groundwater within the NHS system will be monitored from preconstruction to during and continued to post construction, with a frequency of minimum twice a month and will last for three years.

As described above, there will be a comprehensive program in place to monitor groundwater level and the potential impact on the features of the NHS during the proposed development construction. The need for mitigation will be triggered, during the dewatering, excavation, and substructures construction period, when sedimentation, erosion, groundwater level reduction or flooding would be observed within the receiving features of the NHS system.

8.0 REFERENCES

1. Ontario Ministry of the Environment. *Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act.* April15, 2011.
2. Ontario Ministry of Northern Development and Mines. *Quaternary Geology of Ontario - Southern Sheet,* Map 2556, 1991.
3. Ontario Ministry of Northern Development and Mines. *Bedrock Geology of Ontario - Southern Sheet,* Map 2544, 1991.
4. D.K. Todd, *Groundwater Hydrology,* 2nd Edition, John Wiley & Sons, New York, 1980.
5. L.W. Mays, *Water Resources Engineering*, 1st Edition, John Wiley & Sons, New York, 2001.
6. R.F. Craig, *Soil Mechanics*, 7th Edition, Spon Press, London, 2004.
7. MCR Report titled: Preliminary *Geotechnical Report,* R*esidential Development, Northwestern Corner of Regional Road 25 and Britannia Road, Milton, Ontario,* prepared for Mattamy Homes Canada, dated January 2023.
8. MCR Report titled: *Geohydrology Assessment, Residential Development, Northwestern Corner of Regional Road 25 and Britannia Road, Milton, Ontario, prepared for Mattamy Homes Canada,* dated January 2023.
9. Ontario Ministry of the Natural Resources, *Stormwater Management Planning and Design Manual, Section 53 of the Ontario Water Resources Act., Queen's Printer for Ontario, 2003.*
10. *Conservation Authority Guidelines for Development Applications, Hydrogeological Assessment Submissions, June 2013.*

9.0 STATEMENT OF LIMITATIONS

MCR Engineers, Ltd. (MCR) conducted the work associated with this report in accordance with the scope of services, time and budget limitations imposed for this work. The work has been conducted according to reasonable and generally accepted local standards for an environmental consultant at the time of the work. No other warranty or representation, expressed or implied, is included or intended in this report.

The work was designed to provide an overall assessment of the environmental conditions at the Site. The conclusions presented in this report are based on the information obtained during the investigation. The work is intended to reduce the client's risk with respect to environmental impairment. No work can completely eliminate the possibility of further environmental impairment on the Site.

It should be noted that subsurface conditions might vary at locations and depths other than those locations where borings, surveys or explorations were made by MCR. Other contaminants, not tested for in this work, may also potentially be present on the Site. Even with exhaustive investigation, it is not possible to warranty the Site will be free of contaminants. Should conditions, not observed during the work, become apparent, MCR should be immediately notified to assess the situation and conduct additional work, where required. The findings of this report are based on conditions as they were observed at the time of the work.

No assurance is made regarding changes in conditions subsequent to the time of the work. Remediation cost estimates is based on the available information. The estimated costs for remediation only represent the costs for the clean-up of known contaminants that have been identified during the work. Additional costs may be incurred as a result of other contaminants or areas of contamination identified by subsequent work.

Regulatory statutes are subject to interpretation. These statutes and their interpretation may change over time; thus, these issues should be reviewed with appropriate legal counsel.

MCR relied on information provided by others in this report. MCR cannot guarantee the accuracy, completeness and reliability of the information provided by others, although MCR staff attempted to seek clarification on information provided and verifies authenticity, where practical.

The report and its attachments were prepared for and made available for the sole use of the client. MCR will not be responsible for any use or interpretation of the information contained in this report by any other party without the prior expressed written consent of MCR.

10.0 CLOSURE

In accordance with your request and authorization, MCR Engineers Ltd. completed this Geohydrology Assessment Report. This report presented the methodology, findings, and conclusions of the investigation. The Statement of Limitations for all work performed as part of this investigation is included.

We trust that the information provided in this report is sufficient for your present requirements. Should you have any further questions, please do not hesitate to contact our office. Thank you for retaining MCR Engineers Ltd. for this project.

Respectfully,
MCR ENGINEERS LTD.

LICENSED PROFESSIONAL ENGINEER
Z. XIA
PROVINCE OF ONTARIO

Ron Xia, Ph.D., P.Eng.

Date of Issue: July 24, 2023
Updated on: January 18, 2024

APPENDIX

D

Water Quality Unit Sizing Report

D-1 Imbrium Jellyfish Sizing Report and Supporting documents

STANDARD OFFLINE
Jellyfish Filter Sizing Report

Project Information

Date	Friday, January 12, 2024
Project Name	Framgard - South Block Phase 1 and 2
Project Number	231-00962-00
Location	Milton

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.

Jellyfish® Filter

Performance

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

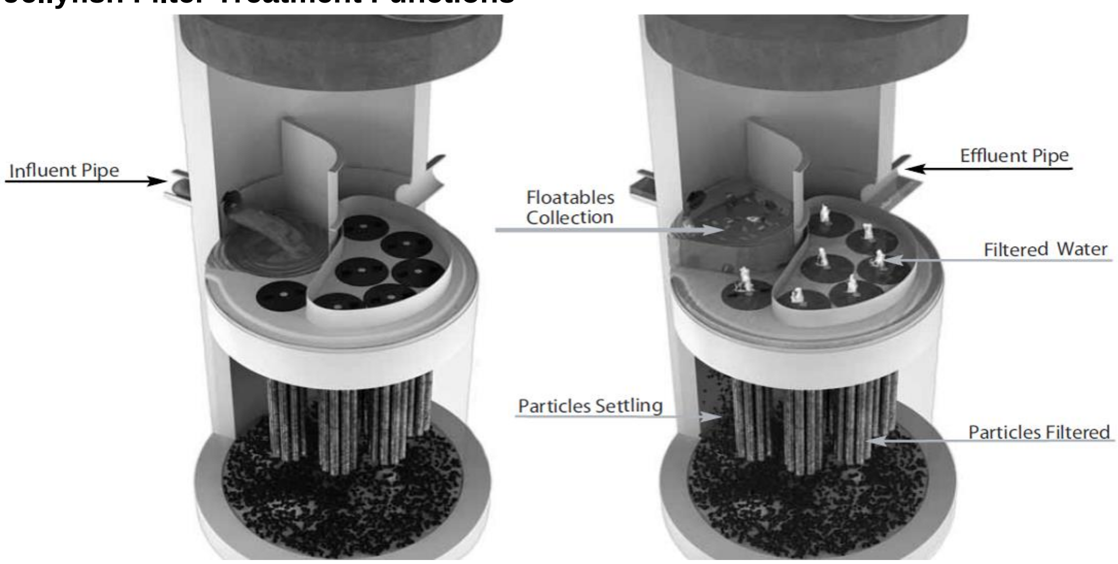
- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 77% TP removal & 51% TN removal
- ☑ 90% Total Copper, 81% Total Lead, 70% Total Zinc
- ☑ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ☑ Free oil, Floatable trash and debris

Field Proven Peformance

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

- A median TSS removal efficiency of 90%, 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 77%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Friday, January 12, 2024
Project Name:	Framgard - South Block Phase 1 and 2
Project Number:	231-00962-00
Location:	Milton

Designer Information

Company:	WSP Canada Group Ltd.
Contact:	Sukhjot Hans
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:	0.894335 ha
Runoff Coefficient:	0.81

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:	**21.3 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 4643 m³, with a suspended sediment concentration of 60 mg/L.	**279 kg***

* Indicates that sediment loading is the limiting parameter in the sizing of this Jellyfish system.

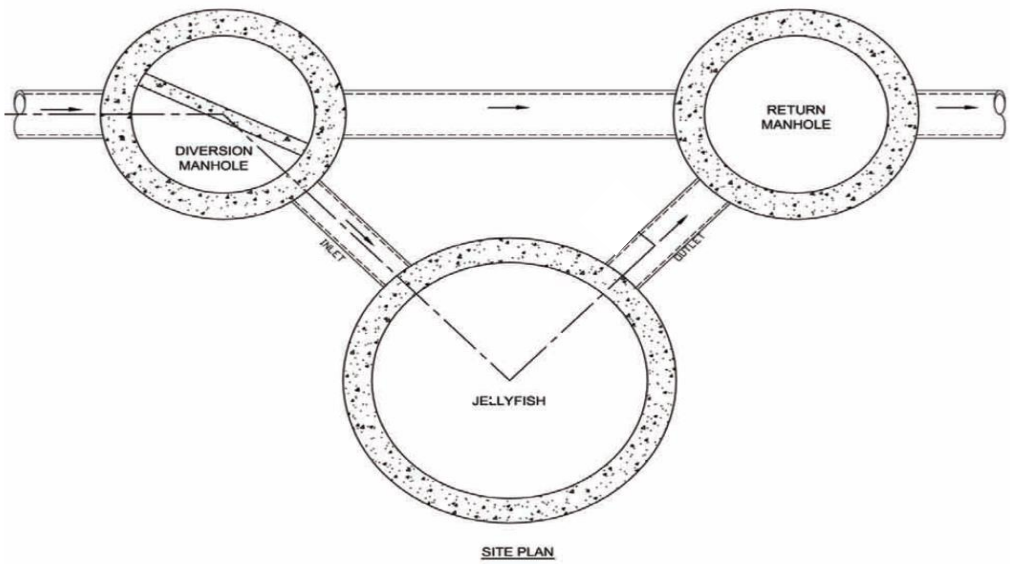
Recommendation

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)	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/ft^2 (0.142 lps/m^2).

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/ft^2 (0.142 lps/m^2).

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

STANDARD OFFLINE
Jellyfish Filter Sizing Report

Project Information

Date	Friday, January 12, 2024
Project Name	Framgard - South Block Phase 3 and 4
Project Number	231-00962-00
Location	Milton

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.

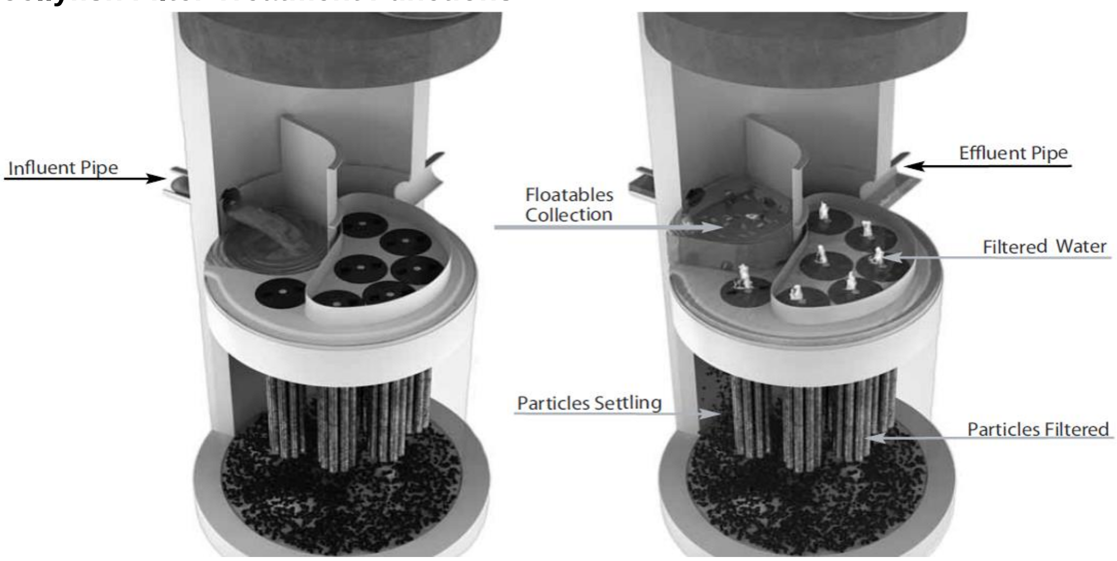
Performance
Jellyfish efficiently captures a high level of Stormwater pollutants, including:

- ☑ 89% of the total suspended solids (TSS) load, including particles less than 5 microns
- ☑ 77% 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 TAPE qualifying rain events and field monitored according to the TAPE field test protocol, demonstrating:

- A median TSS removal efficiency of 90%, 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 77%, and a median Total Nitrogen removal of 51%.

Jellyfish Filter Treatment Functions



Pre-treatment and Membrane Filtration

Jellyfish® Filter

Project Information

Date:	Friday, January 12, 2024
Project Name:	Framgard - South Block Phase 3 and 4
Project Number:	231-00962-00
Location:	Milton

Designer Information

Company:	WSP Canada Group Ltd.
Contact:	Sukhjot Hans
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:	0.967286 ha
Runoff Coefficient:	0.76

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:	**21.5 L/s**
Sediment Loading	Treating 90% of the average annual runoff volume, 4622 m³, with a suspended sediment concentration of 60 mg/L.	**277 kg***

* Indicates that sediment loading is the limiting parameter in the sizing of this Jellyfish system

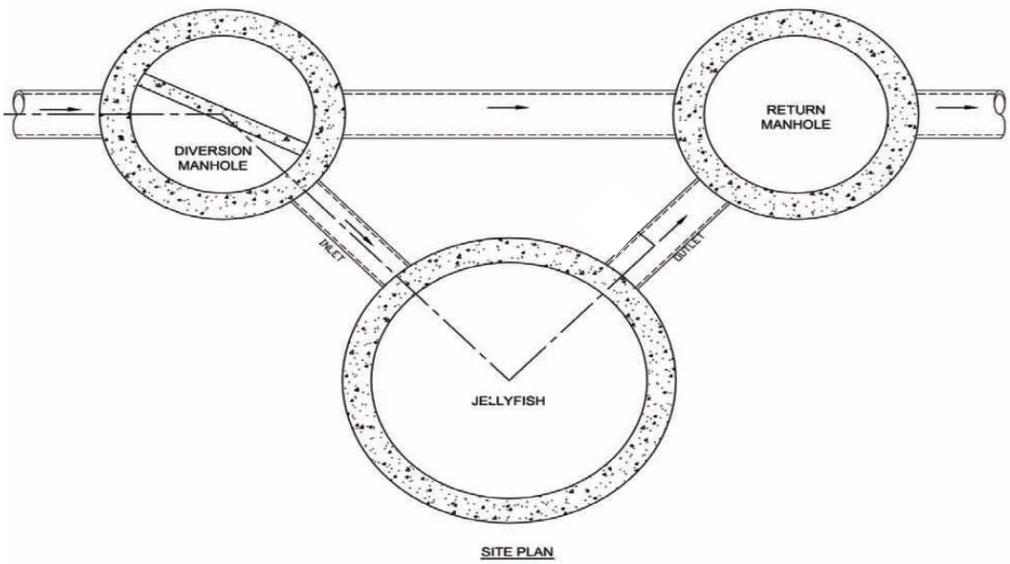
Recommendation

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)	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.
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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.

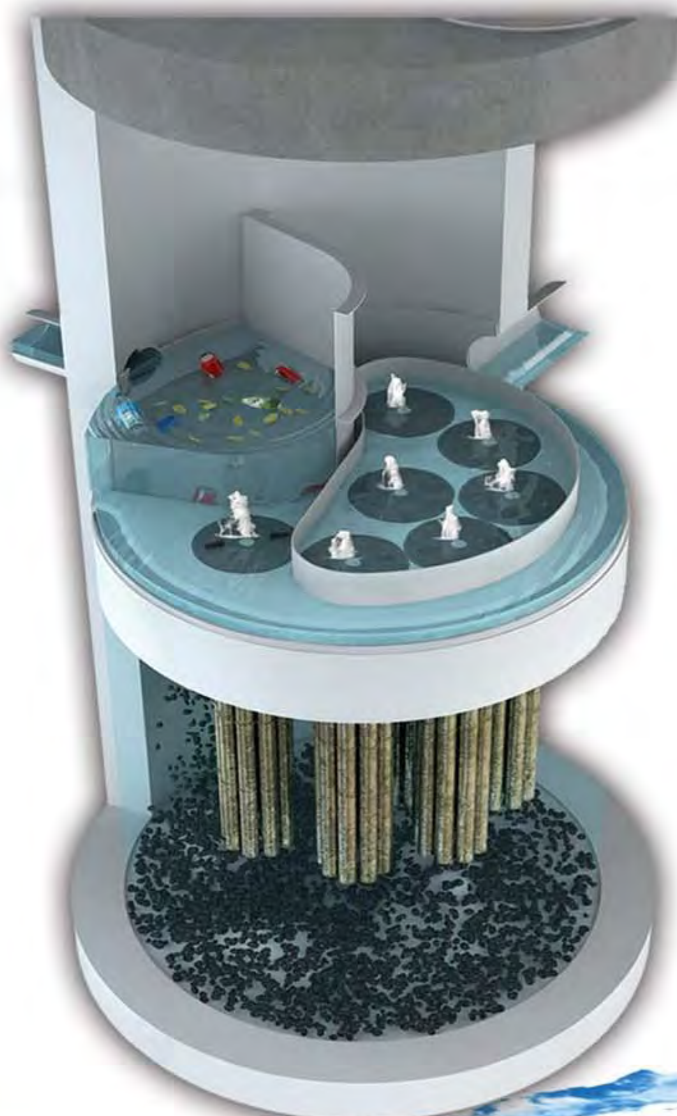
5.2.2 After construction and installation, and during operation, the device shall be inspected and cleaned as necessary based on the manufacturer's recommended inspection and maintenance guidelines and the local regulatory agency/body.

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

Jellyfish® Filter

Technical Manual



imbrium®

Engineered Stormwater Treatment

www.imbriumsystems.com

Jellyfish® Filter

Highlights

The Jellyfish Filter (patent pending) 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.

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

- ✓ Greater than 85% of the total suspended solids (TSS) load, including particles less than 5 microns
- ✓ Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons and bacteria
- ✓ Free oil
- ✓ Floatable trash and debris

Jellyfish® Filter Technical Manual

USA Imbrium Systems
7564 Standish Place, Suite 112
Rockville, MD 20855

Phone: 301.279.8827
Toll Free: 888.279.8826

Canada/International Imbrium Systems Inc.
2 St. Clair Avenue W., Suite 2100
Toronto, ON, M4V 1L5

Phone: 416.960.9900
Toll Free: 800.565.4801

www.imbriumsystems.com

Jellyfish® Filter Quick Glance

The Jellyfish Filter (patent pending) is an engineered stormwater quality treatment technology featuring unique membrane filtration in a compact standalone 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 suspended sediment removal capacity.

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

- Greater than 85! of the total suspended solids (TSS) load, including particles less than 5 microns
- Particulate bound pollutants such as nutrients, toxic metals, hydrocarbons, and bacteria
- Free oil
- Floatable trash and debris

Jellyfish cartridges are passively backwashed automatically after each storm event, which removes accumulated sediment from the membranes and significantly extends the service life of the cartridges and the maintenance interval. If required, the cartridges can be easily manually backwashed without removing the cartridges. Additionally, the lightweight cartridges can be removed by hand and externally rinsed, and rinsed cartridges then reinstalled. These simple maintenance options allow for cartridge regeneration, thereby minimizing cartridge replacement costs and lifecycle treatment costs while ensuring long term treatment performance.

Jellyfish® Filter Patent Information

Jellyfish Filter is protected by one or more of the following patents: Australian Patent No. 2008,286,748; US Patent Nos. 8,123,935, 8,287,726, US 8,221,618; Canadian Patent No. 2,696,482; Korean Patent No. 10-1287539; New Zealand Patent Nos. 583461, 604227; South African Patent No. 201,001,068; Other International Patents Pending.

CANADA/INTL: 800.565.4801 **USA:** 888.279.8826

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Chapter 1

1.0 Imbrium® Systems Contact Information

Imbrium® Systems is an engineered stormwater treatment company that designs, develops, manufactures, and distributes post-construction stormwater quality treatment technologies, to protect water resources from pollutants. Imbrium has a strong record of environmental innovation in the industry as the creator of the Stormceptor® oil and sediment separator, the Jellyfish® Filter, Sorbtive®MEDIA, Sorbtive®FILTER, and Sorbtive®VAULT.

Imbrium Systems is a global company with U.S. headquarters (Imbrium Systems Corporation) located in Rockville, Maryland and Canadian and International headquarters (Imbrium Systems Incorporated and Imbrium International Limited) located in Toronto, Ontario, Canada.

The Jellyfish® Filter is represented by a variety of licensees and organizations globally.

For assistance, please contact Imbrium Systems at:
United States: 888-279-8826 or 301-279-8827
Canada / International: 800-565-4801 or 416-960-9900

Chapter 2

2.0 Jellyfish Filter Design Overview

This technical manual provides information for design and installation of the Jellyfish Filter. When designed properly in accordance with this Technical Manual, the Jellyfish Filter will exceed the performance and longevity of conventional horizontal bed and granular media filters. Test data is available from Imbrium Systems upon request.

2.1 Jellyfish Filter Description

The Jellyfish Filter (patent pending) 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 consists of multiple membrane-encased filter elements ("filtration tentacles") attached to a cartridge head plate. The filtration tentacles provide an extraordinarily large amount of surface area, resulting in superior flow capacity and suspended sediment removal capacity.

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

- Greater than 85% of the total suspended solids (TSS) load, including particles less than 5 microns
- Particulate-bound pollutants such as nutrients, toxic metals, hydrocarbons, and bacteria
- Free oil
- Floatable trash and debris

Jellyfish cartridges are passively backwashed automatically after each storm event, which removes accumulated sediment from the membranes and significantly extends the service life of the cartridges and the maintenance interval. If required, the cartridges can be easily manually backwashed without removing the cartridges. Additionally, the lightweight cartridges can be removed by hand and externally rinsed, and rinsed cartridges then re-installed. These simple maintenance options allow for cartridge regeneration, thereby minimizing cartridge replacement costs and life-cycle treatment costs while ensuring long-term treatment performance. The Jellyfish Filter is comprised of several structural and functional components:

- A **cylindrical (manhole) or rectangular structure** constructed of either precast concrete or fiberglass, and available in a wide variety of sizes and configurations, serves as a vessel that provides long-lasting structural support for the system; provides hydraulic connections to the inlet and outlet pipes; provides surfaces for structural attachment of the cartridge deck and maintenance access wall; provides influent water storage and flow-through volume for pollutant separation and membrane filtration treatment; and provides a high-volume sump for storage of accumulated sediment.

- A rigid high-strength fiberglass **cartridge deck** separates the vessel into a lower chamber and upper chamber; houses the filter cartridges; provides a surface and flow path for treated water to the effluent pipe; provides doublewall containment of oil and other hydrocarbons below deck; and provides a platform for maintenance personnel to safely service the filter cartridges. The lower chamber provides influent water storage and flow-through volume for pollutant separation and membrane filtration treatment, and storage of accumulated sediment. The upper chamber provides above-deck clearance for inspection and maintenance service. The cartridge deck is securely attached to the vessel wall.

- A rigid high-strength fiberglass **maintenance access wall** attenuates influent water velocity; channels influent water into the lower chamber via a large opening in the cartridge deck; provides storage volume for floatable pollutants; and serves as a convenient inspection and maintenance access point for pollutant removal.

- **Cartridge receptacles** are secured to the cartridge deck and together with the cartridge lids, serve to securely anchor the filter cartridges into the cartridge deck.

- **Jellyfish membrane filtration cartridges** are inserted into the cartridge receptacles and secured with the cartridge lids. The filter cartridges treat the influent stormwater by filtering out fine suspended particulates (TSS) and particulate-bound pollutants on the membrane of each filtration tentacle. Filtered water passes through the membranes, flows up the center tube of each filtration tentacle and exits the top opening of each tentacle. Cartridges are available in various lengths and flow ratings. Filter cartridges are designated as either **hi-flo cartridges** or **draindown cartridges**, depending on their placement position within the cartridge deck. Cartridges placed within the backwash pool weir are automatically passively backwashed after each storm event, and are designated the hi-flo cartridges. Cartridges placed outside the backwash pool weir are not passively backwashed but facilitate the draindown of the backwash pool and these are designated the draindown cartridges. The design flow rate of a draindown cartridge is controlled by a cartridge lid orifice to one-half the design flow rate of a hi-flo car tridge of similar length. The lower design flow rate of the draindown cartridge reduces the likelihood of occlusion prior to scheduled maintenance.

- **Cartridge lids** are fastened onto the cartridge receptacles to securely anchor the filter cartridges into the cartridge deck. The lids are removable to allow manual backflushing or removal of the filter cartridges when required during maintenance service. Cartridge lids contain a **flow control orifice** that is specifically sized for use with hi-flo and draindown cartridges. **Blank lids** have no orifice and are used to cover unoccupied cartridge receptacles in systems that do not use the full rated flow capacity of the system.

- A **separator skirt** serves as a baffle that encloses the filtration tentacles and defines the filtration zone inside the separator skirt perimeter. The separator skirt extends the full length of the filtration tentacles and prevents contamination of the membranes with oil and floatable debris. The separator skirt has a large opening at the bottom that allows pre-treated water to enter the filtration zone under low velocity. The separator skirt is securely attached to the underside of the cartridge deck.

- A rigid fiberglass **backwash pool weir** extends 6 inches (150 mm) above the cartridge deck and encloses the hi-flo cartridges. During inflow, filtered water exiting the hi-flo cartridges forms a pool inside the weir. If sufficient driving head is available the pool overtops the weir and spills to the cartridge deck where it subsequently flows to the outlet pipe. As the inflow event subsides and forward driving head decreases, water in the backwash pool reverses flow direction and automatically passively backwashes the hi-flo cartridges, cleaning the membrane surfaces. Water in the lower chamber (below deck) is displaced through the draindown cartridges. This self-cleaning mechanism may occur multiple times during a single storm event as rainfall/runoff intensities rise and fall, thereby significantly extending the service life of the cartridges and the maintenance interval.

- **Optional internal bypass pressure relief pipe(s)** can be placed in one or multiple cartridge receptacles. The pressure relief pipe height and diameter can be varied to accommodate the design peak flow rate and system driving head requirements. When the internal bypass option is utilized, peak flow rates receive membrane filtration treatment up to the filtration design flow rate, with the balance of the peak flow receiving pre-treatment.

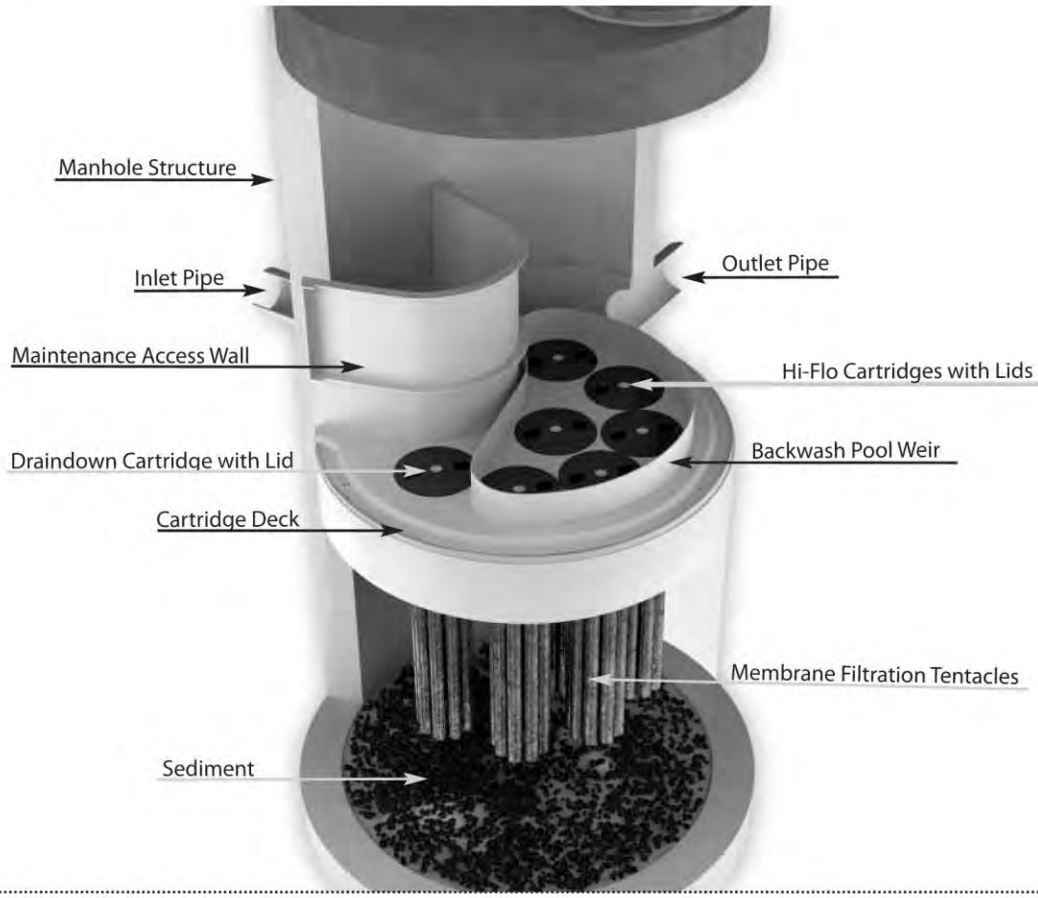
- A **deflector plate** (below-deck inlet pipe manhole configuration only) is installed across the below-deck inlet pipe opening to induce tangential water flow through the pre-treatment channel between the vessel wall and separator skirt.

- **Standard covers, rectangular hatches, or inlet grates** are installed at the surface and are removed to allow maintenance access to the system.

- **Built-in steps or ladder(s)** allow maintenance personnel to access the cartridge deck and filter cartridges. The Jellyfish Filter and components are depicted in **Figure 1.**

FIGURE 1

Jellyfish Filter and Components



Note: Separator Skirt Not Shown

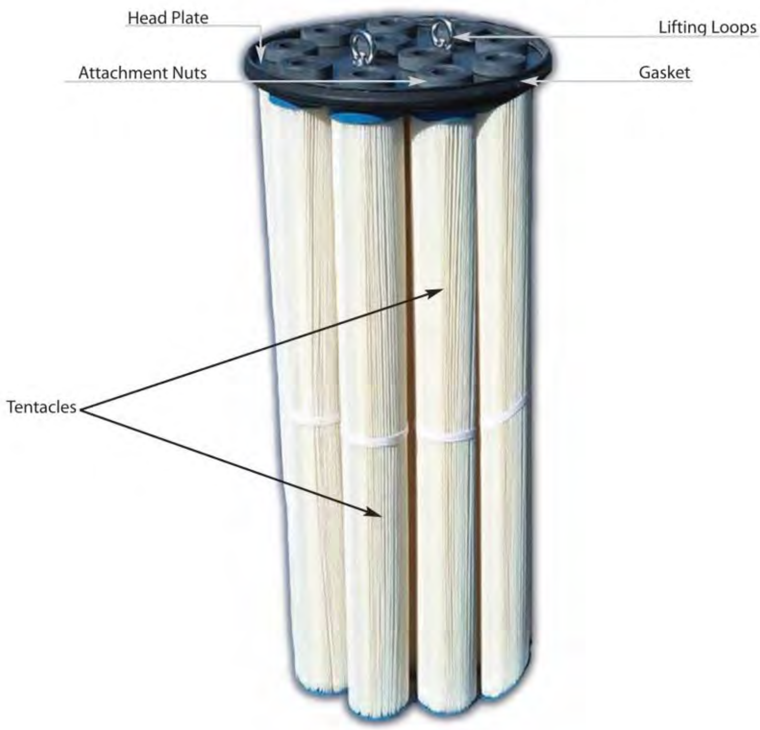
2.2 Jellyfish Membrane Filtration Cartridge

The Jellyfish Filter utilizes multiple lightweight membrane filtration cartridges. Each cartridge consists of multiple removable filter elements ("filtration tentacles") attached to a cartridge head plate. Each filtration tentacle consists of a central perforated tube surrounded by a specialized membrane. The cylindrical filtration tentacle has a threaded pipe nipple at the top and is sealed at the bottom with an end cap. A cluster of tentacles is attached to a stainless steel head plate by inserting the top pipe nipples through the head plate holes and securing with removable nuts. A removable oil-resistant polymeric rim gasket is attached to the head plate to impart a watertight seal when the cartridge is secured into the cartridge receptacle with the cartridge lid. A Jellyfish membrane filtration cartridge is depicted in **Figure 2**.

The cartridge length is typically either 27 inches (686 mm) or 54 inches (1372 mm), with options for custom lengths if required. The dry weight of a new cartridge is less than 20 pounds (9 kg), and the wet weight of a used cartridge is less than 50 pounds (23 kg), making a cartridge easy to install and remove by hand. No heavy lifting equipment is required.

FIGURE 2

Jellyfish Membrane Filtration Cartridge



The filtration tentacle membranes provide an extraordinarily large amount of surface area, resulting in superior flow capacity and suspended sediment removal capacity. A typical Jellyfish cartridge with eleven 54-inch (1372 mm) long filtration tentacles has 381 ft2 (35.4 m2) of membrane surface area. Hydraulic testing on a clean 54-inch (1372 mm) filter cartridge has demonstrated a flow rate of 180 gpm (11.3 L/s) at 18 inches (457 mm) of driving head.

Extensive third-party field testing, including testing at an urban site with very high intensity rainfall and runoff, has demonstrated consistently high pollutant removal performance with a conservative design flow rate of 80 gpm (5.0 L/s) for the 54-inch (1372 mm) long hi-flo cartridge and 40 gpm (2.5 L/s) for the 54-inch (1372 mm) long draindown cartridge. These values translate to a conservative design membrane filtration flux rate (flow per unit surface area) of 0.21 gpm/ft^2 (0.14 Lps/m^2) for the hi-flo cartridge and 0.11 gpm/ft^2 (0.07 Lps/m^2) for the draindown cartridge.

The standard membrane demonstrates removal of >85% of fine sediment at a design flux rate of 0.21 gpm/ft^2, based on laboratory testing with Sil-Co-Sil™106 which has a median particle size (d50) of 22 microns. In addition, the filtration tentacle membrane has anti-microbial characteristics that inhibit the growth of bio-film that might otherwise prematurely occlude the pores of the membrane and restrict hydraulic conductivity.

Hydraulic and sediment loading testing has demonstrated scalability of the membrane filtration surface area such that increases in the number and/or length of filtration tentacles contribute a uniform increase in total filter surface area and therefore flow capacity and sediment removal capacity. The flow rating of a particular Jellyfish Filter cartridge is based on the membrane filtration surface area of the cartridge and data collected from both laboratory testing and field testing.

The cartridge deck contains a receptacle for each filter cartridge. The cartridge is lowered down into the receptacle such that the cartridge head plate and rim gasket rest on the lip of the receptacle. A cartridge lid is fastened onto the receptacle to anchor the cartridge. Each cartridge lid contains a flow control orifice. The orifice in the hi-flo cartridge lid is larger than the orifice in the draindown cartridge lid.

Jellyfish Filter cartridges are designated as either hi-flo cartridges or draindown cartridges, depending on their placement position within the cartridge deck. Cartridges placed within the 6-inch (150 mm) high backwash pool weir that extends above the deck are automatically passively backwashed after each storm event and are designated as the hi-flo cartridges. Cartridges placed outside the backwash pool weir are not passively backwashed but facilitate the draindown of the backwash pool, and these are designated as the draindown cartridges. The design flow rate of a draindown cartridge is controlled by a cartridge lid orifice to one-half the design flow rate of a hi-flo cartridge of similar length. The lower design flow rate of the draindown cartridge reduces the likelihood of occlusion prior to scheduled maintenance.

Inflow events with driving head ranging from less than 1 inch (25 mm) up to the maximum design driving head will cause continuous forward flow and filtration treatment through the draindown cartridges. Inflow events with driving head that exceeds the 6-inch (150 mm) height of the backwash pool weir will cause continuous forward flow and filtration treatment through the hi-flo cartridges.

2.3 Jellyfish Filter Operation – Driving Head Requirement

A differential in upstream and downstream water elevation during an inflow event provides the minimal driving head required to overcome the minor cumulative friction loss through the system, at which point flow-through operation of the Jellyfish Filter commences.

For systems using an external bypass with upstream diversion structure, the driving head is calculated as the difference in elevation between the top of the diversion structure weir and the invert of the Jellyfish Filter outlet pipe. For systems using an internal bypass, the driving head is calculated as the difference in elevation between the top of the pressure relief pipe(s) and the invert of the outlet pipe.

A minimum design driving head is selected to achieve design flow rates, while accounting for gradual increase in system head loss at the design flow rate due to long-term accumulation of sediment on the filtration membranes. A clean Jellyfish Filter cartridge has flow capacity far in excess of the cartridge design flow rate at the design driving head. This ensures that design flow capacity is maintained during the period between maintenance service operations. Typically, a minimum 18 inches (457 mm) of driving head is designed into the system but may vary from 12 to 24 inches (305 to 610 mm) depending on specific site requirements.

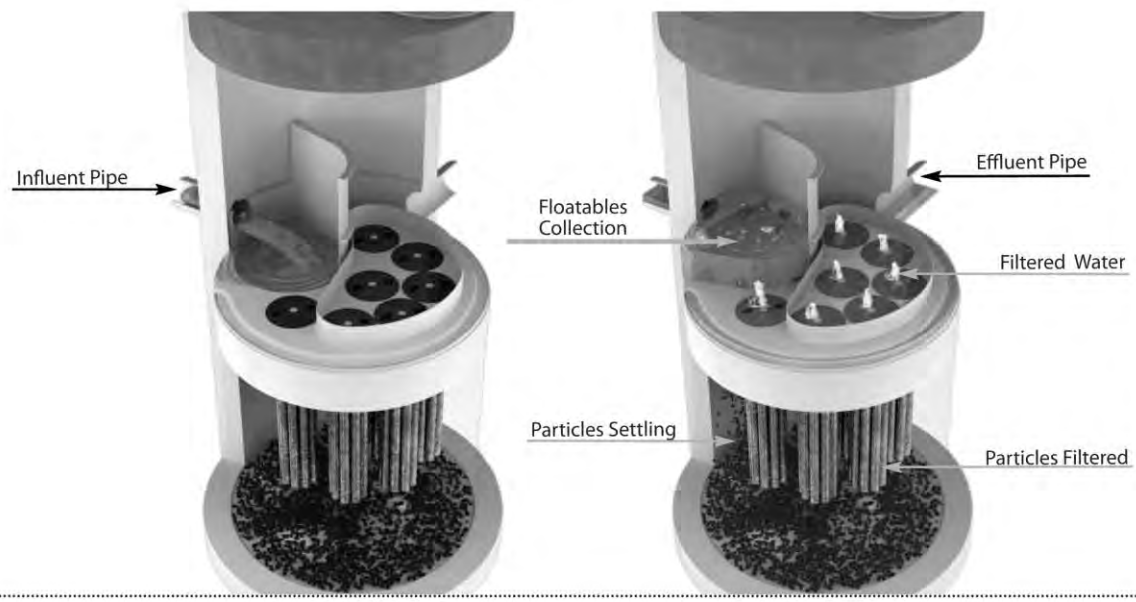
For systems that may experience submerged or backwater conditions due to dry weather base flow or tidal effects, driving head calculations must account for water elevation during the backwater condition. The Jellyfish Filter treatment functions will continue to operate during forward flow despite backwater conditions. An increase in the maintenance access wall height may be required to ensure floatables capture an increase in the height of the backwash pool weir may be required to ensure function of the automatic passive backwash feature.

2.4 Jellyfish Filter Operation – Treatment Functions

The Jellyfish Filter provides both **pre-treatment** and **membrane filtration** treatment to remove pollutants from stormwater runoff. These functions are depicted in **Figure 3** below.

FIGURE 3

Jellyfish Filter Treatment Functions
Pre-Treatment and Membrane Filtration
Section View with Maintenance Access Wall (MAW) Cutaway



Pre-treatment removes coarse sediment (generally > 50 microns), particulate-bound pollutants attached to coarse sediment (nutrients, toxic metals, hydrocarbons), free oil and floatable trash and debris. These pollutants are removed by gravity separation. Large, heavy particles fall to the sump (sedimentation) and low density pollutants rise to the surface (floatation) within the pre-treatment channel.

Pre-treatment begins when influent flow enters the system either through an above-deck inlet pipe (standard) or below-deck inlet pipe (optional). In the above-deck inlet pipe configuration, influent enters the maintenance access wall zone and is channeled through a large-diameter opening in the cartridge deck to the lower chamber. The large surface area of the deck opening and change in flow direction attenuate the influent flow velocity. Due to equalization of hydrostatic pressure and downstream pathway through the opening at the bottom of the separator skirt, influent flow spreads in lateral and downward directions throughout the pre-treatment channel between the vessel wall and the outer perimeter of the separator skirt. In the below-deck inlet pipe configuration, a deflector plate angled across the inlet pipe opening induces directional tangential flow in the pre-treatment channel. In either configuration, flow spreading throughout the pre-treatment channel serves to reduce the average flow velocity and enhance the separation of pollutants.

Pre-treatment for floatables occurs as buoyant pollutants rise toward the surface, with some of the floatables mass trapped beneath the cartridge deck in the pre-treatment channel. Most of the floatables mass accumulating in the maintenance access wall zone at the air-water interface. This feature allows convenient and easy inspection and maintenance for floatable contaminants. The separator skirt protects the filtration tentacles from contamination by oil and floatable debris.

Coarse sediment settles out of the pre-treatment channel to the sump. As water from the pre-treatment channel slowly flows downward and then laterally beneath the separator skirt, the combination of the large opening in the bottom of the separator skirt and a change in direction to an upward downstream flow path serves to further reduce average flow velocity and enhance particle separation. Sediment is stored in the sump until removed by vacuum during a maintenance service.

Membrane filtration treatment removes suspended particulates (generally < 50 microns) and particulate-bound pollutants (nutrients, toxic metals, hydrocarbons, and bacteria). Laboratory and field performance testing of the Jellyfish Filter have demonstrated capture of particulates as small as 2 microns.

Filtration treatment begins when pre-treated influent flows under the separator skirt and into the filtration zone through the large opening defined by the bottom edge of the separator skirt. Uniform hydraulic pressure gradient across the entire membrane surface area causes pre-treated water to penetrate the entire membrane surface area of each filtration tentacle. Water enters the membrane pores radically and deposits fine particulates on the exterior membrane surface. Filtered water flows into the perforated center drain tube of each filtration tentacle and then upward and out the top of each tentacle. Water exiting each of the tentacles of a single cartridge combines at the top of the cartridge under the cartridge lid. The combined flow then vertically exits the cartridge lid orifice with a pulsating fountain effect.

As a layer of sediment builds up on the external membrane surface, membrane pores are partially occluded which serves to reduce the effective pore size. This process, referred to as “filter ripening”, significantly improves the removal efficiency of pollutants relative to a brand new or clean membrane of some nominal pore size. Filter ripening accounts for the ability of the Jellyfish Filter to remove particles finer than the nominal pore size rating of the membranes.

Jellyfish Filter operation and maintenance are depicted in an animation on Imbrium Systems website (www.imbriumsystems.com).

2.5 Jellyfish Filter Operation – Self- Cleaning Functions

The Jellyfish Filter utilizes several self-cleaning processes to remove accumulated sediment from the external surfaces of the filtration membranes, including **automatic passive backwash** of the h i-flo cartridges, **vibrational pulses**, and **gravity**. Combined, these processes significantly extend the cartridge service life, maintenance interval and reduce life-cycle costs.

Automatic passive backwash is performed on the hi-flo cartridge at the end of each runoff event and can also occur multiple times during a single storm event as intensity and driving head varies. During inflow, filtered water exiting the hi-flo cartridges forms a pool above the cartridge deck inside the backwash pool weir. The depth and volume of the back wash pool will vary with the available driving head, ranging from some minimal quantity up to a quantity sufficient to fill and overflow the backwash pool (typical weir height is 6 inches / 150 mm). As the inflow event subsides and forward driving head decreases, water in the backwash pool reverses flow direction and automatically passively backwashes the hi-flo cartridges, removing sediment from the membrane surfaces. Water in the lower chamber (below deck) is displaced through the draindown cartridges.

Vibrational pulses occur as a result of complex and variable pressure and f low direction conditions that a rise in the space between the top surface of the cartridge head plate and the underside of the cartridge lid. During forward flow a stream of filtered water exits the top of each filtration tentacle into this space and en counters resistance from the cartridge lid and turbulent pool of water within the space. Water is forced through the cartridge lid flow control orifice with a pulsating fountain effect. The variable localized pressure causes pulses that transmit vibrations to the membranes, thereby dislodging accumulated sediment. The effect appears more pronounced at higher flow rates, and applies to both hi-flo and draindown cartridges.

Gravity continuously applies a force to ac cumulate d sediment on the membranes, both during inflow events and inter-event dry periods. As fine particles agglomerate into larger masses on the membrane surface, adhesion to the membrane surface can lessen, and a peeling effect ensues which ultimately results in agglomerates falling away from the membrane. Complex chemical and biological effects may also play a role in this process.

Chapter 3

3.0 Jellyfish Filter Design Guidelines

The Jellyfish Filter has many flexible design features to accommodate a wide range of specific site requirements and constraints. For design assistance, please contact Imbrium Systems.

3.1 Configurations and Design Capacities

Design flow capacities and pollutant capacities for standard Jellyfish Filter manhole configurations are shown in **Tables 1** and **2**.

The Jellyfish Filter standard model numbers provide information about the manhole inside diameter (expressed in U.S. customary units) and cartridge counts for hi-flo and draindown cartridges. For example, Jellyfish Filter Model Number JF6-4-1 is a 6-ft (1.8 m) diameter manhole with four hi-flo cartridges and one draindown cartridge. Standard model numbers assume the use of 54-inch (1372 mm) long cartridges. Specific designations for non-standard structures or cartridge lengths are noted in the **Jellyfish Filter Owner's Manual**.

Table 1
Design Flow Capacities
Standard Jellyfish Filter Manhole Configurations

Manhole Diameter (ft / m) [1]	Model No.	Hi-Flo Cartridges [2] 54 in / 1372 mm	Draindown Cartridges [2] 54 in / 1372 mm	Treatment Flow Rate (gpm / cfs)	Treatment Flow Rate (L/S)
4 / 1.2	JF4-2-1	2	1	200 / 0.45	12.6
6 / 1.8	JF6-3-1	3	1	280 / 0.62	17.7
	JF6-4-1	4	1	360 / 0.80	22.7
	JF6-5-1	5	1	440 / 0.98	27.8
	JF6-6-1	6	1	520 / 1.16	32.8
8 / 2.4	JF8-6-2	6	2	560 / 1.25	35.3
	JF8-7-2	7	2	640 / 1.43	40.4
	JF8-8-2	8	2	720 / 1.60	45.4
	JF8-9-2	9	2	800 / 1.78	50.5
	JF8-10-2	10	2	880 / 1.96	55.5
10 / 3.0	JF10-11-3	11	3	1000 / 2.23	63.1
	JF10-12-3	12	3	1080 / 2.41	68.1
	JF10-12-4	12	4	1120 / 2.50	70.7
	JF10-13-4	13	4	1200 / 2.67	75.7
	JF10-14-4	14	4	1280 / 2.85	80.8
	JF10-15-4	15	4	1360 / 3.03	85.8
	JF10-16-4	16	4	1440 / 3.21	90.8
	JF10-17-4	17	4	1520 / 3.39	95.9
	JF10-18-4	18	4	1600 / 3.56	100.9
	JF10-19-4	19	4	1720 / 3.83	108.5
12 / 3.6	JF12-20-5	20	5	1800 / 4.01	113.6
	JF12-21-5	21	5	1880 / 4.19	118.6
	JF12-22-5	22	5	1960 / 4.37	123.7
	JF12-23-5	23	5	2040 / 4.54	128.7
	JF12-24-5	24	5	2120 / 4.72	133.8
	JF12-25-5	25	5	2200 / 4.90	138.8
	JF12-26-5	26	5	2280 / 5.08	143.8
	JF12-27-5	27	5	2360 / 5.26	148.9

[1] Smaller and larger systems may be custom designed
[2] Shorter length cartridge configurations are available

Table 2
Design Pollutant Capacities
Standard Jellyfish Filter Manhole Configurations

Model Diameter (ft / m)	Wet Volume Below Deck (ft^3 / L)	Sediment Capacity [1] (ft^3 / L)	Oil Capacity [2] (gal / L)
JF4 4 / 1.2	82 / 2313	12 / 0.34	100 / 379
JF6 6 / 1.8	184 / 5205	28 / 0.79	224 / 848
JF8 8 / 2.4	327 / 9252	50 / 1.42	388 / 1469
JF10 10 / 3.0	511 / 14,456	78 / 2.21	608 / 2302
JF12 12 / 3.6	735 / 20,820	113 / 3.20	732 / 2771

[1] Assumes 12 inches (305 mm) of sediment depth in sump.
Systems may be designed with increased sediment capacity.
[2] Assumes 24 inches (610 mm) of pre-treatment channel depth for oil storage

3.2 Inlet and Outlet Pipes

The Jellyfish Filter is available in both the standard **above-deck inlet pipe** configuration and optional **below-deck inlet** pipe configuration. Specific site requirements generally determine the configuration that is most favorable for the site. For both configurations, the invert elevation of the outlet pipe is identical to the cartridge deck elevation. Please refer to **Figures 4** and **5**.

FIGURE 4

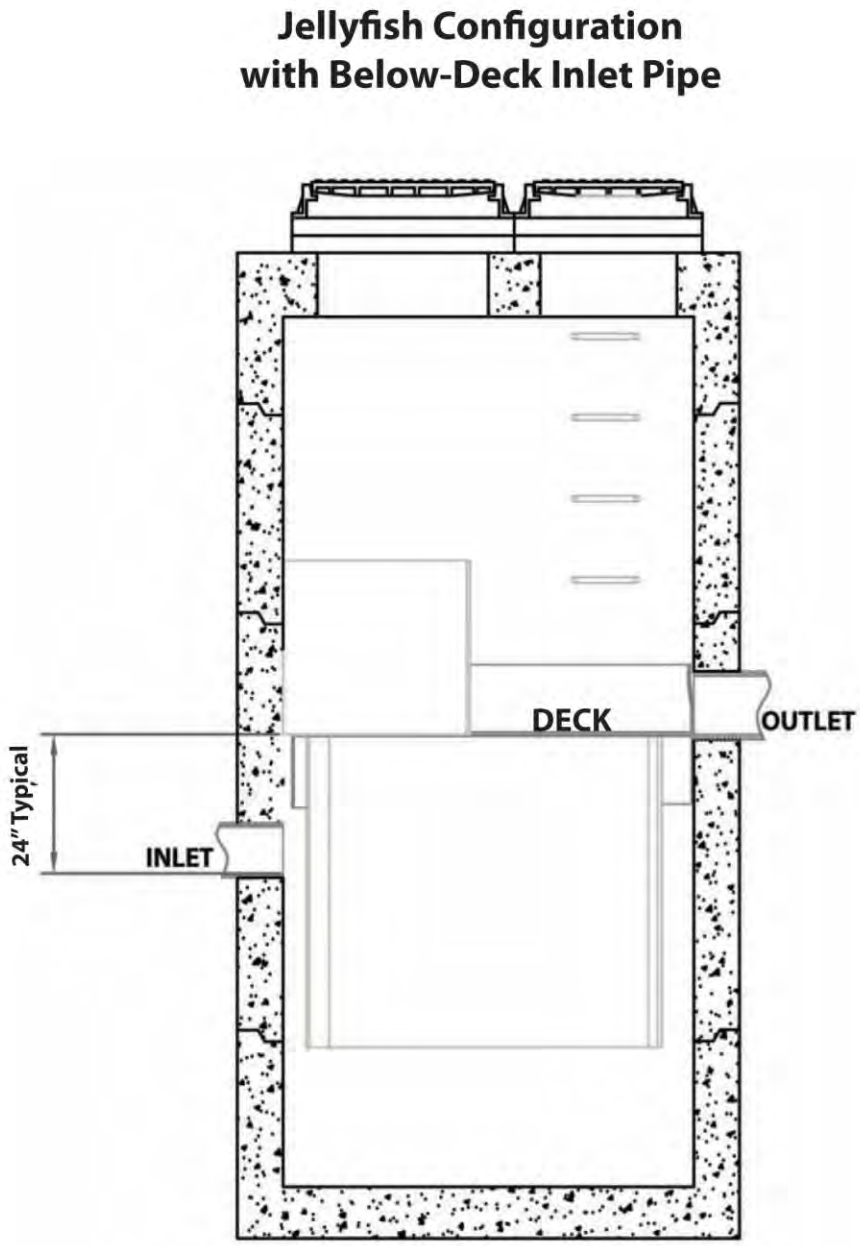
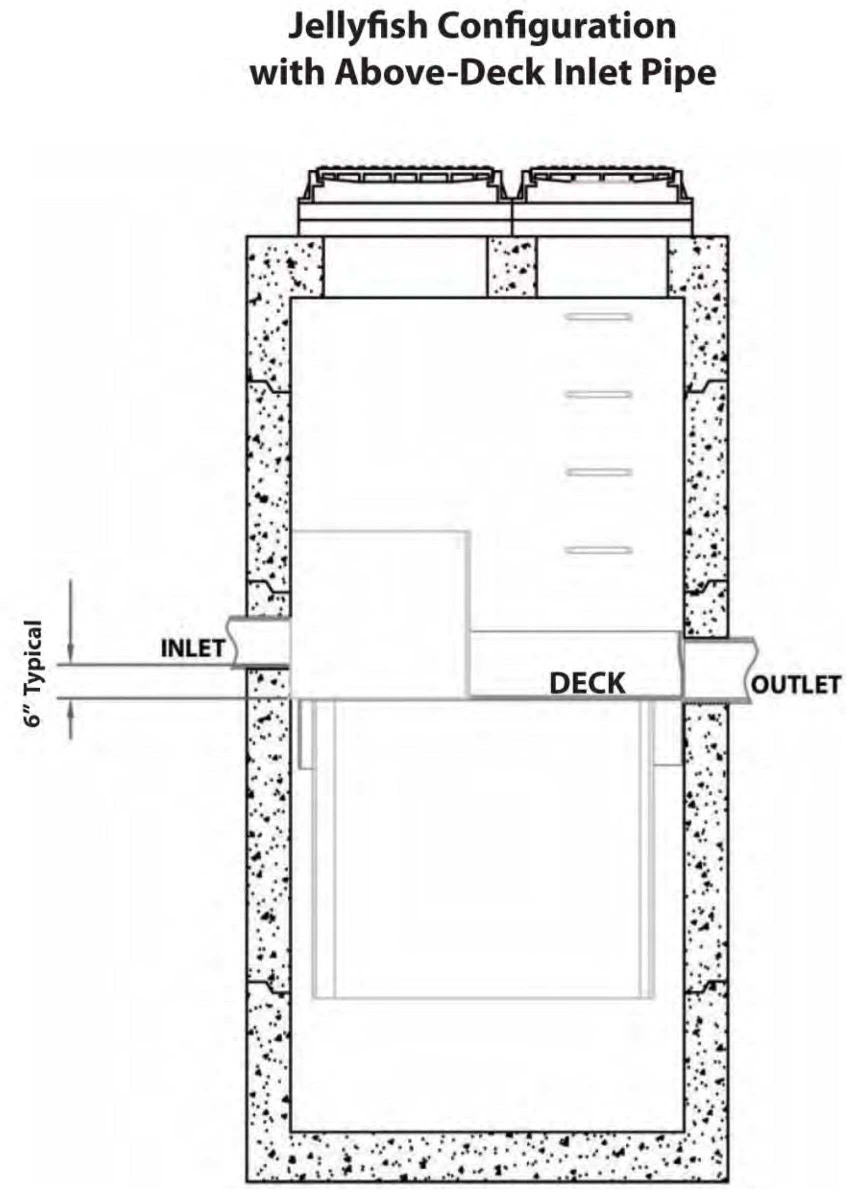

Jellyfish Configuration
with Below-Deck Inlet Pipe
DECK
OUTLET
24" Typical
INLET

FIGURE 5


Jellyfish Configuration
with Above-Deck Inlet Pipe
6" Typical
INLET
DECK
OUTLET

For the standard above-deck inlet pipe configuration, the invert elevation of the inlet pipe is typically set 6 inches (150 mm) higher than the invert elevation of the outlet pipe. This generally ensures that the inlet pipe will drain completely at the conclusion of each rainfall/runoff event, while providing sufficient volume within the maintenance access wall zone for surface accumulation of floatables below the inlet pipe. The elevation of the inlet pipe can be varied as required.

The Jellyfish Filter can accommodate a wide range of angles between the inlet and outlet pipes. The inlet pipe can be located anywhere about the circumference of the structure. The separation angle relationship of the inlet pipe to the outlet pipe can vary from 0 to 360 degrees to provide maximum design flexibility. Typical off-line layouts (external bypass using an upstream diversion structure) will have an inlet to outlet separation angle of 90 to 120 degrees. See **Table 3** below for the minimum separation angle for standard manhole configurations with an above-deck inlet pipe.

The Jellyfish Filter can accommodate **multiple inlet pipes** within certain restrictions.

The Jellyfish Filter can be built at all depths of cover generally associated with conventional stormwater conveyance systems.

Table 3
Minimum Inlet and Outlet Pipe Separation Angles and Diameters
(Jellyfish Filter Manhole Configurations with Above-Deck Inlet Pipe)

Model Diameter (ft / m)	Minimum Angle [1] Inlet / Outlet Pipes	Minimum Inlet Pipe Diameter (in / mm)	Minimum Outlet Pipe Diameter (in / mm)
JF4 4 / 1.2	62°	6 / 152	8 / 203
JF6 6 / 1.8	59°	8 / 203	10 / 254
JF8 8 / 2.4	52°	10 / 254	12 / 305
JF10 10 / 3.0	48°	12 / 305	18 / 457
JF12 12 / 3.6	40°	12 / 305	18 / 457

[1] Assumes off-line (external bypass) configuration

3.3 Bypass Design

The Jellyfish Filter can be designed with either an off-line or on-line configuration. All stormwater filter systems will perform for a longer duration between required maintenance services when designed and applied in off-line configurations.

A standard off-line configuration has an external bypass that uses an upstream diversion structure. The elevation difference between the top of the diversion structure weir and the Jellyfish Filter outlet pipe invert establishes the design driving head associated with the design flow rate. Excess flow that overtops the diversion weir bypasses the Jellyfish Filter and proceeds downstream. Drawings that illustrate relative system elevations are available by contacting Imbrium Systems.

For some sites an off-line configuration may not be practical and use of an on-line configuration is advantageous. In these cases, an optional internal bypass pressure relief pipe(s) can be placed in one or multiple cartridge receptacles within the Jellyfish Filter. The pressure relief pipe height and diameter can be varied to accommodate the design peak flow rate and system driving head requirements. For these systems the driving head is calculated as the difference in elevation between the top of the pressure relief pipe and the invert of the outlet pipe. When the internal bypass option is utilized, peak flow rates receive membrane filtration treatment up to the filtration design flow rate, with the balance of the peak flow receiving pre-treatment. Increased sump depth may be required to increase sediment storage capacity and to minimize re-suspension of previously captured sediment at peak flow rates. Please contact Imbrium Systems for design assistance.

3.4 Shallow or Low Cover Installations

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 to provide adequate access to all the cartridges within the unit. The general minimum depth of cover is 36 inches (915 mm) from the Jellyfish outlet pipe invert to the underside of the top slab. Further custom modifications may be possible. A typical drawing is included in **Appendix A** and **B**.

3.5 Submerged Installations

When properly designed, the Jellyfish Filter will function effectively under submerged conditions. For systems that may experience submerged or backwater conditions due to dry weather base flow or tidal effects, driving head calculations must account for water elevation during the backwater condition. The Jellyfish Filter treatment functions will continue to operate during forward flow despite backwater conditions. A customized increase to the maintenance access wall height may be required to ensure floatables capture and an increase in the height of the backwash pool weir may be required to ensure function of the automatic passive backwash feature.

3.6 Grated Inlet and Curb Inlet Jellyfish Filters

Existing drainage systems can be retrofitted by replacing conventional storm inlets with a Jellyfish Filter inlet. Imbrium Systems has two standard options, curb inlet and grated inlet configurations. Both configurations utilize the shorter 27-inch (686 mm) length Jellyfish filter cartridges and require minimal cover. Two typical drawings are included in **Appendix A** and **B**. Further custom modifications may be possible.

3.7 Series Jellyfish Filter

For sites with water quality treatment flow rates that exceed the design flow rate of the largest standard Jellyfish Filter model, custom systems can be designed that hydraulically connect multiple Jellyfish Filters in series. Please contact Imbrium Systems for assistance.

3.8 Jellyfish Filter with Sump Drain

The Jellyfish Filter is typically designed to maintain a pool of water in the lower chamber (below deck) between storms. However, certain sites or jurisdictions may require draindown of the sump between storms. To meet these requirements, a sump drain filter can be installed to slowly drain the lower chamber pool to the sub-grade for infiltration or to an alternate point of discharge. A typical drawing is included in **Appendix A** and **B**.

Chapter 4

4.0 Jellyfish Filter Sizing Guidelines

The Jellyfish Filter is sized based on considerations of the specified treatment flow rate, anticipated sediment mass load transported from the site and required pollutant storage capacities.

An optional software-based continuous simulation modeling tool, such as **PCSWMM for Stormceptor®**, can be utilized to determine site hydrology from local historical rainfall data and thereby assist in sizing a Jellyfish Filter. In general, such a tool is useful in deriving the water quality treatment flow rate associated with treatment of a high percentage of the average annual runoff volume.

4.1 Sizing for Water Quality Treatment Flow Rate

The Jellyfish Filter can be sized using a specified flow rate (i.e. “water quality flow rate” or “treatment flow rate”). The treatment flow rate is determined by the engineer in accordance with methods approved by the local jurisdiction. The appropriate Jellyfish Filter model number is then selected from **Table 1**. Custom systems can be designed for sites with water quality treatment flow rates that exceed the design flow rate of the largest standard Jellyfish Filter model. Please contact Imbrium Systems for assistance.

4.2 Sizing for Sediment Mass Loading

A second sizing consideration is the anticipated sediment load that will enter the Jellyfish Filter. For a stormwater filter system to have practical application in the field, it is important that the system’s sediment mass loading and storage is recommended that a system be designed to accommodate a minimum one year interval between maintenance services for pollutant removal and filter cartridge flushing/rinsing.

Laboratory testing using a standard test sediment demonstrated sediment mass loading capacity of 125 pounds (57 kg) of sediment per 54-inch (1372 mm) long hi-flo cartridge at 18 inches (457 mm) of driving head (see **Table 4** below). Specific site conditions will influence the sediment mass loading capacity of the Jellyfish Filter due to the variable nature of sediment characteristics, rainfall intensity, time intervals between runoff events and frequency of automatic passive backwash.

The projected annual sediment load transported from the site should be determined by the engineer. Calculations can be performed for the projected annual runoff volume using an assumed event mean suspended solids concentration (typically 60 mg/L for urban sites). As a guideline, the U.S. EPA has determined typical annual sediment loads per acre for various sites by land use (see **Table 5**). Certain states and local jurisdictions have also established such guidelines.

For some sites the Jellyfish Filter is installed downstream of a detention facility. In these cases, the Jellyfish Filter will typically treat a relatively low flow rate (orifice-controlled release flow rate) from the detention facility compared to flow rates that would be treated if the Jellyfish Filter received the site runoff directly. In such cases, the size of the Jellyfish Filter and number of filter cartridges will typically be determined by the projected annual sediment mass load transported to the Jellyfish Filter, accounting for sediment mass that is expected to settle out in the upstream detention facility.

It is important for the engineer to confirm that the system design has adequate storage capacity for anticipated pollutant loads that will accumulate over the specified maintenance interval. The oil and sediment pollutant capacities for each standard Jellyfish Filter model are shown in **Table 2**.

Table 4
Sediment Mass Loading Capacity
Jellyfish Filter Hi-Flo and Draindown Cartridges

Cartridge Type	Cartridge Length (in / mm)	Driving Head (in / mm)	Sediment Mass Loading Capacity [1, 2] (lbs / kg)
Hi-Flo	27 / 686	18 / 457	63 / 28
Hi-Flo	54 / 1372	18 / 457	125 / 57
Draindown	27 / 686	18 / 457	32 / 15
Draindown	54 / 1372	18 / 457	63 / 28

[1] Based on laboratory testing using simulated storm events and Sil-Co-Sil™ 106 test sediment (d_{50} = 22 microns) at 40% of maximum cartridge flow rate

[2] Sediment Mass Loading Capacity expressed as pounds of NJPSD test sediment (1 – 1000 microns, d_{50} = 67 microns, characterized as 55% sand / 40% silt / 5% clay), using conversion factor of 1.66 from Sil-Co-Sil 106 to NJPSD

Note: Actual sediment mass loading capacity will vary depending on specific site characteristics

Table 5
Typical Urban Areas and Pollutant Yields (Sediment)
(Burton and Pitt, 2002)

Annual Total Suspended Solids Load by Land Use (lbs/acre/year) (kg/hectare/year)							
Commercial	**Parking Lot**	**Residential Density**			**Highways**	**Industrial**	**Shopping Centers**
		High	Med.	Low			
1000	400	400	250	10	880	500	440
1120	448	448	280	11	986	560	493

Source: U.S. EPA Stormwater Best Management Practice Design Guide, Volume 1, Appendix D, Table D-1, Burton and Pitt 2002

4.3 Continuous Simulation Sizing Tool

A software-based continuous simulation modeling tool such as **PCSWMM for Stormceptor®**, can be utilized to determined site hydrology from local historical rainfall data, thereby assist in sizing a Jellyfish Filter. In general, such a tool is useful in deriving the water quality treatment flow rate associated with treatment of a high percentage (typically 80 - 90%) of the average annual runoff volume. The appropriately sized Jellyfish Filter is then selected from **Table 1** based on the derived water quality treatment flow rate. Please contact Imbrium Systems for assistance with optional sizing methodology.

Chapter 5

5.0 Jellyfish Filter Installation

The installation of the precast concrete or fiberglass Jellyfish Filter structure should conform to state highway, provincial or local specifications for the installation of maintenance manholes. Selected sections of a general specification that are applicable are summarized in the following sections.

For more information, please refer to the Jellyfish Filter Performance Specification with document title **Standard Specification – Water Quality Filter Treatment Device**.

<u>Excavation</u>

- Excavation and general site preparation for the installation of the Jellyfish Filter structure should conform to state highway, provincial or local specifications.
- Topsoil removed during the excavation should be stockpiled in designated areas and should not be mixed with subsoil or other materials.
- The Jellyfish Filter structure should not be installed on frozen ground.
- Excavation should extend a minimum of 12 inches (300 mm) from the precast concrete surfaces plus an allowance for shoring and bracing where required.
- If the bottom of the excavation provides an unsuitable foundation additional excavation may be required. In areas with a high water table, continuous dewatering may be required to ensure that the excavation is stable and free of water.

- Level the sub-grade to the proper elevation. Verify the elevation against the structure dimensions, the invert elevations on the approved Jellyfish Filter drawing and the site plans. Adjust the base aggregate if necessary. Verify the soil bearing capacity is adequate for the required load.

Installation of Jellyfish Filter Structure

- Set the base section of the Jellyfish Filter structure on solid sub-grade.
- Verify the level and elevation of the base section before adding any riser sections.
- Add specified watertight seal to the base section. Set riser section(s) on the base section.
- Install the inlet and outlet pipes to the structure.
- Install the top slab and frames and covers.
- Do not install Jellyfish membrane filtration cartridges until the upstream catchment and site have been stabilized.

Installation of Jellyfish Membrane Filtration Cartridges

- After the upstream catchment and site have been stabilized, remove any accumulated sediment and debris from the structure.

- Safely descend to the cartridge deck using the ladder attached to the sidewall of the manhole. Confined space entry procedures are required.

- Carefully lower the Jellyfish membrane filtration cartridges into the cartridge receptacles within the cartridge deck. A filter cartridge should be placed into each of the draindown cartridge receptacles outside the backwash pool weir.

- Depending on the specific Jellyfish Filter model number, filter cartridges should be placed into most or all of the hi-flo cartridge receptacles within the backwash pool weir. If a membrane joint snags on the receptacle lip, use a slight twisting or sideways motion to clear the snag. Do not force the membranes down into the cartridge receptacle, as this may damage the membranes. Use a slight downward pressure on the cartridge head plate to seat the rim gasket (thick circular gasket on the stainless steel head plate) into the cartridge receptacle.

- Examine the cartridge lids to differentiate lids with a small orifice, a large orifice and no orifice. Lids with a small orifice are to be inserted into the draindown cartridge receptacles. Lids with a large orifice are to be inserted into the hi-flo cartridge receptacles. Lids with no orifice are to be inserted into unoccupied cartridge receptacles within the backwash pool weir.

- To install a cartridge lid, ensure the cartridge lid male threads are aligned properly with the cartridge receptacle female threads. Firmly twist the cartridge lid clockwise to seat the filter cartridge snugly in place.

Chapter 6

6.0 Jellyfish Filter Inspection and Maintenance

For inspection and maintenance information, please refer to the **Jellyfish® Filter Owner's Manual**. Jellyfish Filter operation and maintenance are depicted in an **animation** on Imbrium Systems website (**www.imbriumsystems.com**).

Chapter 7

7.0 Jellyfish Filter Replacement Parts

Replacement parts for the JellyfishFilter can be ordered by contacting Imbrium Systems at:

United States: 888-279-8826 or 301-279-8827
Canada / International: 800-565-4801 / 416-960-9900
www.imbriumsystems.com

Chapter 8

8.0 Jellyfish Filter Performance Specification

The Jellyfish Filter Performance Specification is contained in the document titled **Standard Specification – Water Quality Filter Treatment Device**, shown in **Appendix C** and available on Imbrium Systems website.

Jellyfish® Filter Standard Specifications
Water Quality Filter Treatment Device

Part 1- General

1.1 Work Included

Specifies requirements for construction and performance of an underground stormwater quality filter 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 D 4097: Contact Molded Glass Fiber Reinforced Chemical Resistant Tanks
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

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 precast concrete and/or fiberglass (FRP) components.

1.4 Handling and Storage
Prevent damage to materials during storage and handling.

Part 2 - Products

2.1 General

2.1.1

The device shall be circular or rectangular and 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. Alternatively, the device shall be constructed of fiberglass (FRP), installed to conform to applicable sections of state, provincial and municipal building codes, highway, municipal or local specifications for the construction of such devices.

2.1.2 Fiberglass Insert (Cartridge Deck)

The concrete device shall include a fiberglass insert bolted and sealed watertight inside the precast concrete chamber. Alternatively, the fiberglass device shall include a fiberglass insert bolted and/or chemically welded watertight inside the fiberglass chamber. The fiberglass insert 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; (d) a conduit for conveyance of treated water to the effluent pipe.

2.1.3 Membrane Filter Cartridges

Filter cartridges shall be comprised of cylindrical membrane filter elements connected to a perforated head plate. The number of membrane filter elements per cartridge shall be eleven 2.75-inch (70-mm) diameter elements. The length of each filter element shall be a minimum 27 inches (690 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. The maximum treatment flow rate of a filter cartridge shall be controlled by an orifice in the cartridge lid and based on a design flux rate determined by the maximum treatment flow rate per unit of filtration membrane surface area. The maximum flux rate shall be 0.21 gpm/ft2 (0.142 lps/m2). Each lightweight membrane filter cartridge shall allow for manual installation and removal and shall have a dry installation weight not to exceed the following:

Cartridge Length		**Maximum Cartridge Dry Weight for Installation**	
27 inches	690 mm	20 pounds	9 kg
54 inches	1,370 mm	25 pounds	12 kg

2.1.4 Backwashing Cartridges

The filter device shall have a weir extending above the cartridge deck that encloses the high flow rate filter cartridges when placed in their respective cartridge receptacles within the cartridge deck. The weir shall collect a pool of water during inflow events that subsequently automatically backwashes the hi flo rate cartridges when the inflow event subsides. All filter cartridges shall allow for use of a manual backwashing or filtration membrane rinsing procedure to restore flow capacity and sediment capacity and extend cartridge service life.

2.1.5 Maintenance Access to Captured Pollutants

A Maintenance Access Wall shall enclose an opening in the cartridge deck that has minimum diameter of 18 inches (450 mm) and thereby provide suitable access for removal of accumulated floatable pollutants and sediment.

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 66-degrees or less in the stormwater conveyance system.

2.1.7 Double-Wall Containment of Hydrocarbons

The 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. Alternatively, a device constructed of fiberglass (FRP) does not require double-wall containment as fiberglass is resistant to hydrocarbon penetration.

2.1.8 Separator Skirt
The device shall provide a flexible separator skirt that extends from the underside of the cartridge deck to a minimum length equal to the length of the membrane filter elements. The separator skirt shall serve as a baffle to protect the membrane filter elements from contamination by floatables and coarse sediment.

2.1.9 Sump
The device must 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.

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.

2.3 Gaskets
All gaskets used for the concrete joints shall be manufactured using neoprene or nitrile rubber gaskets to prevent deterioration from presence of captured petroleum hydrocarbons. Mastic sealants or butyl tape are not an acceptable alternative as they are prone to leakage of petroleum hydrocarbons.

2.4 Frame and Cover
Frame and covers must be manufactured from cast-iron and embossed with the name of the device manufacturer or the device brand name.

2.5 Doors and Hatches
If provided shall meet designated loading requirements at a minimum for incidental traffic.

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

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

2.7 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.

2.8 Inspections
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 Function
The stormwater quality filter treatment device functions to remove pollutants by the following unit treatment processes; sedimentation, floatation and membrane filtration.

3.1.2 Pollutants
The stormwater quality filter treatment device removes oil, debris, trash, sediment, sediment-bound pollutants, metals and nutrients from stormwater during frequent wet weather events.

3.1.3 Bypass
The stormwater quality filter treatment device typically operates off-line.

3.1.4 Treatment Flux Rate
The stormwater quality filter treatment device shall treat 100% of the required water quality treatment flow based on a maximum treatment flux 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 with a minimum 20 TARP qualifying rain events and field monitoring conducted according to the TARP or TAPE field test protocol.

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

3.2.2 Fine Particle Removal
The stormwater quality filter treatment device shall demonstrate the ability to capture fine particles as indicated by an effluent d50 of 15 microns or lower for all monitored storm events, and an effluent turbidity of 25 NTUs or lower.

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

3.3 Lab Test Performance

3.3.1 Suspended Solids Removal
The stormwater quality treatment device shall demonstrate the ability to remove a minimum of 85% of Sil-Co-Sil 106 (d50 = 22 microns), measured as SSC, with a 95% confidence interval at the system's 100% operating rate with influent sediment concentrations ranging from 100 to 300 mg/L.

3.4 Inspection and Maintenance

The stormwater quality filter device shall have the following features:

3.4.1 The membrane filter elements shall be designed to last three years prior to requiring replacement.

3.4.2 Inspection which includes trash and floatables collection, sediment depth determination, and visible determination of backwash pool depth shall be easily conducted from grade.

3.4.3 Manual backflushing of the filter cartridges shall be possible to restore the flow capacity and sediment capacity of the filter cartridges and therefore extend cartridge service life.

3.4.4 Filter treatment shall have a minimum 12 inches (610 mm) of sediment storage depth.

3.4.5 Sediment removal from the filter treatment device shall be conducted using a standard maintenance truck and vacuum apparatus, and a single point of entry through the cartridge deck that is unobstructed by filter cartridges.

3.4.6 Filter cartridges be easily maintained without the use of additional lifting equipment.

Part 4 – Execution

4.1 Precast & Installation

4.1.1 Construction Sequence

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

The 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
- riser section and/or transition slab (if required)
- maintenance riser section(s) (if required)
- frame and access cover

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.

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. Once the stormwater quality treatment device has been constructed, any lift holes must be plugged watertight with mortar or non-shrink grout.

4.1.2 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.

4.1.3 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 Fiberglass (FRP) Installation

4.2.1 Construction Sequence

The installation of the FRP device should conform to applicable sections of state, provincial and municipal building codes and highway, municipal or local specifications for the construction of such devices. Selected sections of a general specification that are applicable are summarized below. For detailed installation instructions refer to the submitted drawing and installation details.

Structural - Proposed installation details shall conform to all federal, provincial, state, municipal or other local specifications as may be applicable, including all building code requirements.

Water Quality Device Construction Sequence - The water quality FRP device is installed in the following sequence:

- Water quality device as delivered to site placed on prepared bedding or slab using spreader bars and the lifting lugs provided on the structure. Avoid lifting chains or cables from contacting sides of tank
- Do not drop, roll or slide vessel
- Backfill using approved back fill material
- Pour anti-buoyancy slab as required per the drawing
- Connect inlet and outlet pipes
- Riser sections and/or transitions (if required and if shipped separately)
- Frame and access cover

4.2.2 Frame and Cover Installation

No direct structural connection shall be permitted to any FRP maintenance access surface riser pipe. No vertical structural connection shall be permitted to any FRP component under any circumstances unless approved by the manufacturer.

A minimum 1-inch (25 mm) gap shall be left around and above any required FRP maintenance access surface risers (i.e. not a buried installation), with this gap filled with pea gravel or approved fill material against the surrounding structure that must support the frame and cover in its entirety.

4.3 Maintenance Access Wall
In some instances the Maintenance Access Wall will 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.

4.4 Filter Cartridge Installation
Filter cartridges shall be installed in the cartridge deck after the construction site is fully stabilized, unless otherwise specified by the design engineer.

4.5 Filter Cartridge Installation
Manufacturer shall coordinate delivery of filter cartridges and other internal components with contractor. Filter cartridges shall be delivered and installed after site is stabilized and unit is ready to accept cartridges. Contractor shall take appropriate action to protect the filter cartridge receptacles and filter cartridges from damage during construction. For systems with cartridges installed prior to full site stabilization and prior to system commissioning, the contractor can plug inlet and outlet pipes to prevent stormwater from entering the device. Plugs must be removed after the device has been commissioned.

Part 5 – Quality Assurance

5.1 Clean Up and Restoration
Each component of the water quality treatment device shall be inspected by the Owners Representative prior to final acceptance. The contractor shall remove soil and debris created by the storm drainage work from the structure. At the completion of all work, the structure and surrounding area shall be left in a neat, safe and orderly condition.

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.

5.2 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.

Jellyfish® Filter

General Design Notes

✓ Driving Head

- ✓ Typical driving head designed into the system is 18 inches (457 mm)
- ✓ Off-line configuration has an external bypass that uses an upstream diversion structure to provide the 18 inches (457 mm) of driving head

✓ Inlet & Outlet Pipes

- ✓ A wide range of angles can be accommodated
  - The inlet pipe can be located anywhere about the structure circumference
  - The separation angle relationship between the inlet pipe and outlet pipe can vary from 0 to 360 degrees providing maximum design flexibility.
- ✓ Multiple inlet pipes can be accommodated
- ✓ For the standard above-deck configuration, the inlet pipe's invert elevation is typically set 6 inches (150 mm) higher than the invert elevation of the outlet pipe.
  - Alternative invert elevations between the inlet and outlet pipe can easily be accommodated with most all configurations

✓ Minimum Cover

- ✓ Low cover installations generally need a minimum depth of cover of 36 inches (915 mm) from the outlet pipe invert to the underside of the top slab.

✓ Cartridge Details:

- ✓ Cartridge length is typically either 27 inches (686 mm) or 54 inches (1372 mm)
  - Design flow rates for the 54-inch (1372 mm) length:
    - hi-flo cartridge: 80 gpm (5.0 L/s)
    - draindown cartridge: 40 gpm (2.5 L/s)
  - Design membrane filtration flux rate (flow rate per unit surface area) for the 54-inch (1372 mm) length:
    - hi-flo cartridge: 0.21 gpm/ft2 (0.14 Lps/m2)
    - draindown cartridge: 0.11 gpm/ft2 (0.07 Lps/m2)
  - Weight for the 54-inch (1372 mm) length:
    - Dry weight of a new cartridge is < 20 pounds (9 kg)
    - Wet weight of a used cartridge is < 50 pounds (23 kg)

imbrium®
Engineered Stormwater Treatment

USA
Imbrium Systems
7564 Standish Place, Suite 112
Rockville, MD 20855

Phone: 301.279.8827
Toll Free: 888.279.8826

Canada/International
Imbrium Systems Inc.
2 St. Clair Avenue W., Suite 2100
Toronto, ON M4V 1L5

Phone: 416.960.9900
Toll Free: 800.565.4801

www.imbriumsystems.com

JFTM-V2-0511

D-2 Imbrium Stormceptor EFO and Supporting documents

Stormceptor®

Rinker MATERIALS® A QUIKRETE® COMPANY

Stormceptor® EF Sizing Report

Imbrium® Systems
ESTIMATED NET ANNUAL SEDIMENT (TSS) LOAD REDUCTION

01/13/2024

Province:	Ontario
City:	Milton
Nearest Rainfall Station:	TORONTO INTL AP
Climate Station Id:	6158731
Years of Rainfall Data:	20

Site Name:	North Block Phase 5-8

Drainage Area (ha):	1.713198
Runoff Coefficient 'c':	0.79

Particle Size Distribution:	CA ETV
Target TSS Removal (%):	60.0
Required Water Quality Runoff Volume Capture (%):	90.0

Estimated Water Quality Flow Rate (L/s):	42.09
Oil / Fuel Spill Risk Site?	Yes
Upstream Flow Control?	No
Peak Conveyance (maximum) Flow Rate (L/s):	
Influent TSS Concentration (mg/L):	200
Estimated Average Annual Sediment Load (kg/yr):	1065
Estimated Average Annual Sediment Volume (L/yr):	866

Project Name:	Framgard
Project Number:	231-00962-00
Designer Name:	Brandon O'Leary
Designer Company:	Rinker Pipe
Designer Email:	brandon.oleary@RinkerPipe.com
Designer Phone:	905-630-0359
EOR Name:	Sukjhot Hans
EOR Company:	WSP Canada Group Ltd.
EOR Email:	
EOR Phone:	

Net Annual Sediment (TSS) Load Reduction Sizing Summary

Stormceptor Model	TSS Removal Provided (%)
EFO4	43
EFO6	53
EFO8	59
EFO10	62
EFO12	64

Recommended Stormceptor EFO Model: EFO10

Estimated Net Annual Sediment (TSS) Load Reduction (%): 62

Water Quality Runoff Volume Capture (%): > 90

imbrium

THIRD-PARTY TESTING AND VERIFICATION

►**Stormceptor® EF and Stormceptor® EFO** are the latest evolutions in the Stormceptor® oil-grit separator (OGS) technology series, and are designed to remove a wide variety of pollutants from stormwater and snowmelt runoff. These technologies have been third-party tested in accordance with the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators** and performance has been third-party verified in accordance with the **ISO 14034 Environmental Technology Verification (ETV)** protocol.

PERFORMANCE

►**Stormceptor® EF and EFO** remove stormwater pollutants through gravity separation and floatation, and feature a patent-pending design that generates positive removal of total suspended solids (TSS) throughout each storm event, including high-intensity storms. Captured pollutants include sediment, free oils, and sediment-bound pollutants such as nutrients, heavy metals, and petroleum hydrocarbons. Stormceptor is sized to remove a high level of TSS from the frequent rainfall events that contribute the vast majority of annual runoff volume and pollutant load. The technology incorporates an internal bypass to convey excessive stormwater flows from high-intensity storms through the device without resuspension and washout (scour) of previously captured pollutants. Proper routine maintenance ensures high pollutant removal performance and protection of downstream waterways.

PARTICLE SIZE DISTRIBUTION (PSD)

►The **Canadian ETV PSD** shown in the table below was used, or in part, for this sizing. This is the identical PSD that is referenced in the Canadian ETV ***Procedure for Laboratory Testing of Oil-Grit Separators*** for both sediment removal testing and scour testing. The Canadian ETV PSD contains a wide range of particle sizes in the sand and silt fractions, and is considered reasonably representative of the particle size fractions found in typical urban stormwater runoff.

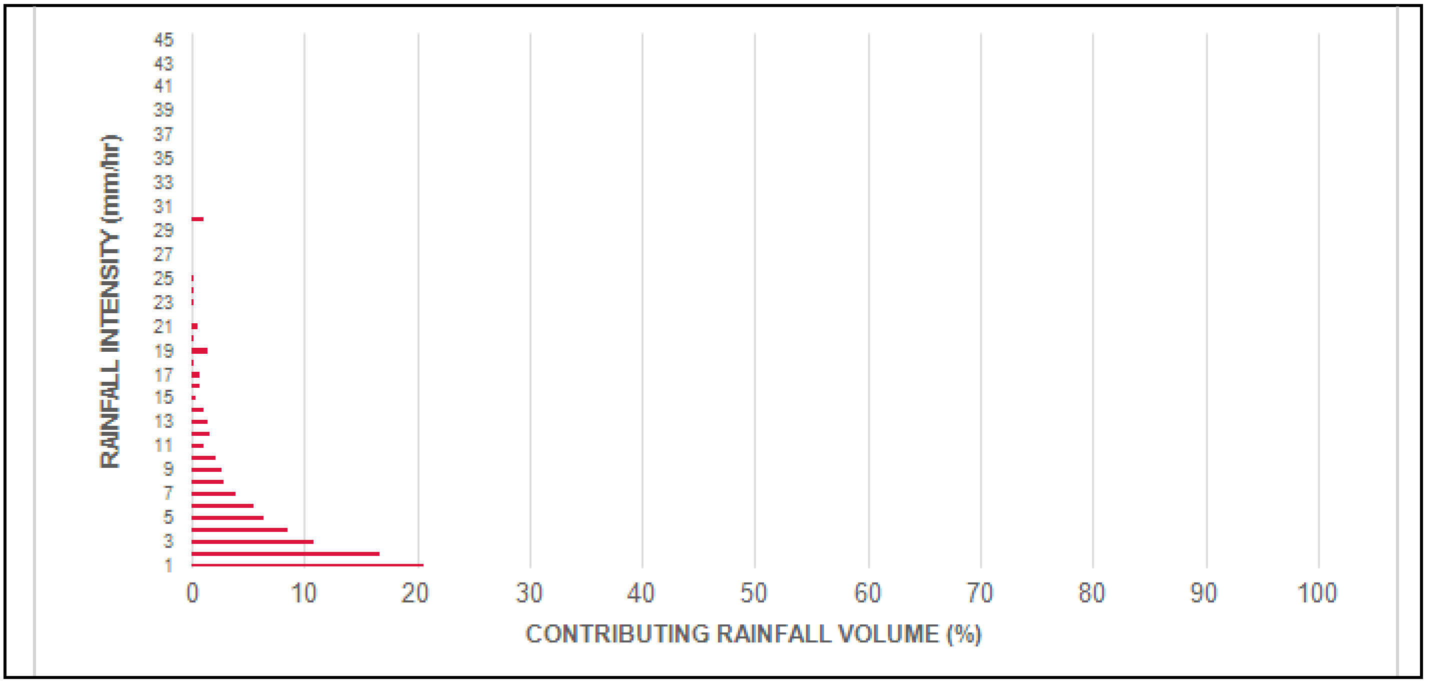
Particle Size (μm)	Percent Less Than	Particle Size Fraction (μm)	Percent
1000	100	500-1000	5
500	95	250-500	5
250	90	150-250	15
150	75	100-150	15
100	60	75-100	10
75	50	50-75	5
50	45	20-50	10
20	35	8-20	15
8	20	5-8	10
5	10	2-5	5
2	5	<2	5

Stormceptor® EF Sizing Report

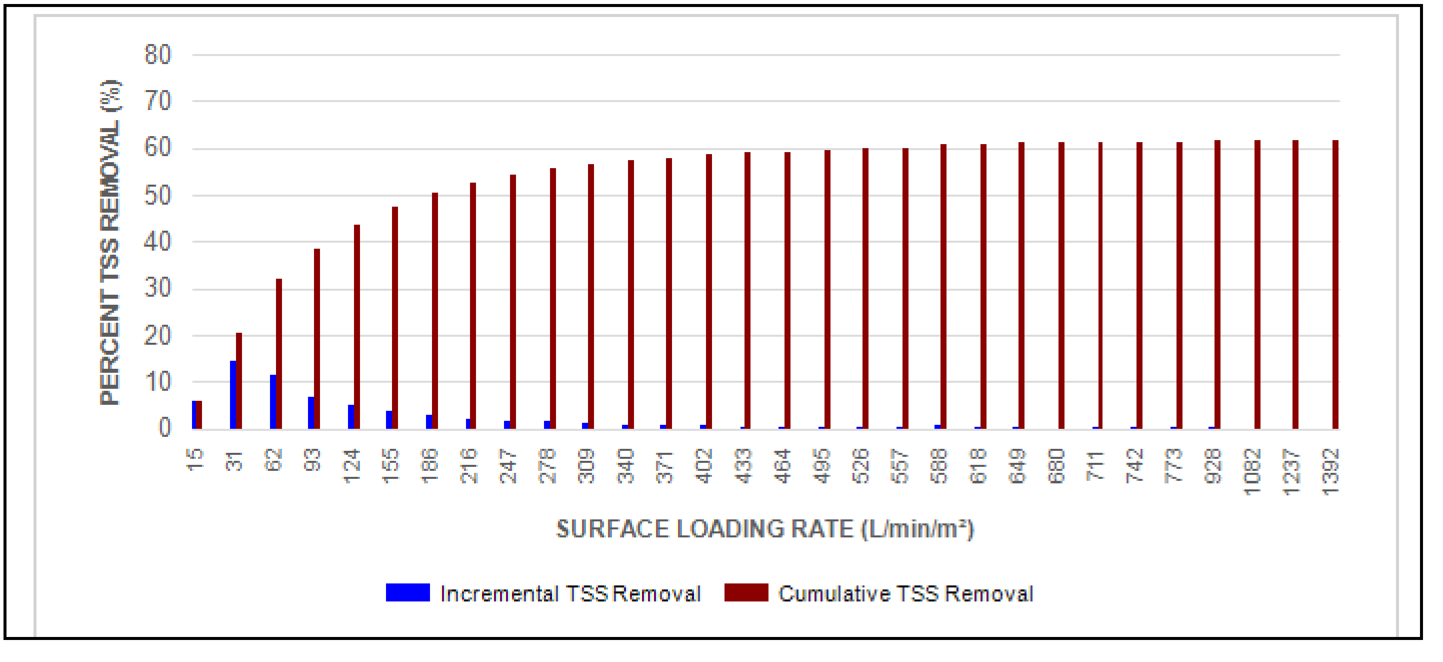
Rainfall Intensity (mm / hr)	Percent Rainfall Volume (%)	Cumulative Rainfall Volume (%)	Flow Rate (L/s)	Flow Rate (L/min)	Surface Loading Rate (L/min/m²)	Removal Efficiency (%)	Incremental Removal (%)	Cumulative Removal (%)
0.50	8.5	8.5	1.88	113.0	15.0	70	6.0	6.0
1.00	20.6	29.1	3.76	226.0	31.0	70	14.5	20.5
2.00	16.8	45.9	7.53	452.0	62.0	67	11.3	31.8
3.00	10.8	56.7	11.29	677.0	93.0	63	6.8	38.6
4.00	8.5	65.2	15.05	903.0	124.0	61	5.1	43.7
5.00	6.4	71.6	18.81	1129.0	155.0	58	3.7	47.5
6.00	5.5	77.0	22.58	1355.0	186.0	56	3.0	50.5
7.00	3.9	81.0	26.34	1580.0	216.0	54	2.1	52.6
8.00	2.9	83.9	30.10	1806.0	247.0	53	1.5	54.1
9.00	2.7	86.5	33.86	2032.0	278.0	52	1.4	55.5
10.00	2.2	88.7	37.63	2258.0	309.0	51	1.1	56.6
11.00	1.0	89.7	41.39	2483.0	340.0	50	0.5	57.1
12.00	1.7	91.3	45.15	2709.0	371.0	49	0.8	57.9
13.00	1.4	92.8	48.91	2935.0	402.0	48	0.7	58.6
14.00	1.0	93.7	52.68	3161.0	433.0	47	0.5	59.1
15.00	0.3	94.0	56.44	3386.0	464.0	46	0.1	59.2
16.00	0.8	94.8	60.20	3612.0	495.0	45	0.4	59.6
17.00	0.8	95.7	63.96	3838.0	526.0	44	0.4	59.9
18.00	0.2	95.8	67.73	4064.0	557.0	44	0.1	60.0
19.00	1.5	97.3	71.49	4289.0	588.0	43	0.6	60.7
20.00	0.2	97.5	75.25	4515.0	618.0	42	0.1	60.7
21.00	0.6	98.2	79.01	4741.0	649.0	42	0.3	61.0
22.00	0.0	98.2	82.78	4967.0	680.0	42	0.0	61.0
23.00	0.2	98.4	86.54	5192.0	711.0	41	0.1	61.1
24.00	0.2	98.6	90.30	5418.0	742.0	41	0.1	61.2
25.00	0.2	98.9	94.06	5644.0	773.0	41	0.1	61.3
30.00	1.1	100.0	112.88	6773.0	928.0	40	0.5	61.7
35.00	0.0	100.0	131.69	7901.0	1082.0	39	0.0	61.7
40.00	0.0	100.0	150.50	9030.0	1237.0	37	0.0	61.7
45.00	0.0	100.0	169.31	10159.0	1392.0	34	0.0	61.7
Estimated Net Annual Sediment (TSS) Load Reduction =								**62 %**

Climate Station ID: 6158731 *Years of Rainfall Data*: 20

RAINFALL DATA FROM TORONTO INTL AP RAINFALL STATION



INCREMENTAL AND CUMULATIVE TSS REMOVAL
FOR THE RECOMMENDED STORMCEPTOR® MODEL



Maximum Pipe Diameter / Peak Conveyance

Stormceptor EF / EFO	Model Diameter		Min Angle Inlet / Outlet Pipes	Max Inlet Pipe Diameter		Max Outlet Pipe Diameter		Peak Conveyance Flow Rate	
	(m)	(ft)		(mm)	(in)	(mm)	(in)	(L/s)	(cfs)
EF4 / EFO4	1.2	4	90	609	24	609	24	425	15
EF6 / EFO6	1.8	6	90	914	36	914	36	990	35
EF8 / EFO8	2.4	8	90	1219	48	1219	48	1700	60
EF10 / EFO10	3.0	10	90	1828	72	1828	72	2830	100
EF12 / EFO12	3.6	12	90	1828	72	1828	72	2830	100

SCOUR PREVENTION AND ONLINE CONFIGURATION

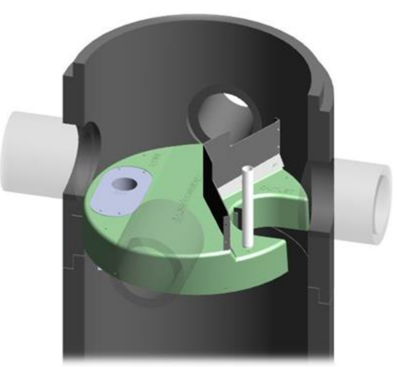
►**Stormceptor® EF and EFO** feature an internal bypass and superior scour prevention technology that have been demonstrated in third-party testing according to the scour testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**, and the exceptional scour test performance has been third-party verified in accordance with the ISO 14034 ETV protocol. As a result, Stormceptor EF and EFO are approved for online installation, eliminating the need for costly additional bypass structures, piping, and installation expense.

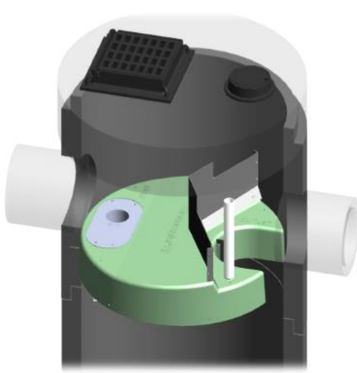
DESIGN FLEXIBILITY

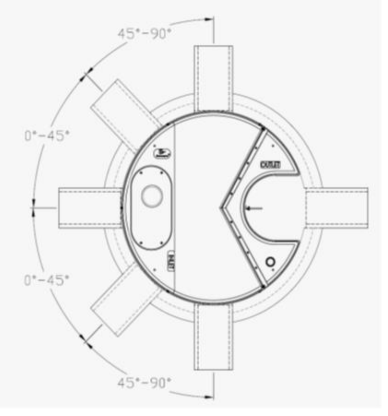
►**Stormceptor® EF** and **EFO** offers design flexibility in one simplified platform, accepting stormwater flow from a single inlet pipe or multiple inlet pipes, and/or surface runoff through an inlet grate. The device can also serve as a junction structure, accommodate a 90-degree inlet-to-outlet bend angle, and can be modified to ensure performance in submerged conditions.

OIL CAPTURE AND RETENTION

►While Stormceptor® EF will capture and retain oil from dry weather spills and low intensity runoff, **Stormceptor® EFO** has demonstrated superior oil capture and greater than 99% oil retention in third-party testing according to the light liquid re-entrainment testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**. Stormceptor EFO is recommended for sites where oil capture and retention is a requirement.







INLET-TO-OUTLET DROP

Elevation differential between inlet and outlet pipe inverts is dictated by the angle at which the inlet pipe(s) enters the unit.
0° - 45° : The inlet pipe is 1-inch (25mm) higher than the outlet pipe.
45° - 90° : The inlet pipe is 2-inches (50mm) higher than the outlet pipe.

HEAD LOSS

The head loss through Stormceptor EF is similar to that of a 60-degree bend structure. The applicable K value for calculating minor losses through the unit is 1.1. For submerged conditions the applicable K value is 3.0.

Pollutant Capacity

Stormceptor EF / EFO	Model Diameter		Depth (Outlet Pipe Invert to Sump Floor)		Oil Volume		Recommended Sediment Maintenance Depth *		Maximum Sediment Volume *		Maximum Sediment Mass **	
	(m)	(ft)	(m)	(ft)	(L)	(Gal)	(mm)	(in)	(L)	(ft^3)	(kg)	(lb)
EF4 / EFO4	1.2	4	1.52	5.0	265	70	203	8	1190	42	1904	5250
EF6 / EFO6	1.8	6	1.93	6.3	610	160	305	12	3470	123	5552	15375
EF8 / EFO8	2.4	8	2.59	8.5	1070	280	610	24	8780	310	14048	38750
EF10 / EFO10	3.0	10	3.25	10.7	1670	440	610	24	17790	628	28464	78500
EF12 / EFO12	3.6	12	3.89	12.8	2475	655	610	24	31220	1103	49952	137875

*Increased sump depth may be added to increase sediment storage capacity
** Average density of wet packed sediment in sump = 1.6 kg/L (100 lb/ft^3)

Feature	Benefit	Feature Appeals To
Patent-pending enhanced flow treatment and scour prevention technology	Superior, verified third-party performance	Regulator, Specifying & Design Engineer
Third-party verified light liquid capture and retention for EFO version	Proven performance for fuel/oil hotspot locations	Regulator, Specifying & Design Engineer, Site Owner
Functions as bend, junction or inlet structure	Design flexibility	Specifying & Design Engineer
Minimal drop between inlet and outlet	Site installation ease	Contractor
Large diameter outlet riser for inspection and maintenance	Easy maintenance access from grade	Maintenance Contractor & Site Owner

STANDARD STORMCEPTOR EF/EFO DRAWINGS

For standard details, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

STANDARD STORMCEPTOR EF/EFO SPECIFICATION

For specifications, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

Table of TSS Removal vs Surface Loading Rate Based on Third-Party Test Results
Stormceptor® EFO

SLR (L/min/m²)	TSS % REMOVAL	SLR (L/min/m²)	TSS % REMOVAL	SLR (L/min/m²)	TSS % REMOVAL	SLR (L/min/m²)	TSS % REMOVAL
1	70	660	42	1320	35	1980	24
30	70	690	42	1350	35	2010	24
60	67	720	41	1380	34	2040	23
90	63	750	41	1410	34	2070	23
120	61	780	41	1440	33	2100	23
150	58	810	41	1470	32	2130	22
180	56	840	41	1500	32	2160	22
210	54	870	41	1530	31	2190	22
240	53	900	41	1560	31	2220	21
270	52	930	40	1590	30	2250	21
300	51	960	40	1620	29	2280	21
330	50	990	40	1650	29	2310	21
360	49	1020	40	1680	28	2340	20
390	48	1050	39	1710	28	2370	20
420	47	1080	39	1740	27	2400	20
450	47	1110	38	1770	27	2430	20
480	46	1140	38	1800	26	2460	19
510	45	1170	37	1830	26	2490	19
540	44	1200	37	1860	26	2520	19
570	43	1230	37	1890	25	2550	19
600	42	1260	36	1920	25	2580	18
630	42	1290	36	1950	24	2600	26

STANDARD PERFORMANCE SPECIFICATION FOR "OIL GRIT SEPARATOR" (OGS) STORMWATER QUALITY TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground Oil Grit Separator (OGS) device for stormwater quality treatment, with third-party testing results and a Statement of Verification in accordance with ISO 14034 Environmental Management – Environmental Technology Verification (ETV).

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental management – Environmental technology verification (ETV)

Canadian Environmental Technology Verification (ETV) Program's **Procedure for Laboratory Testing of Oil-Grit Separators**

1.3 SUBMITTALS

1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.

1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.

1.3.3 Unless directed otherwise by the Engineer of Record, OGS stormwater quality treatment product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

PART 2 – PRODUCTS

2.1 OGS POLLUTANT STORAGE

The OGS device shall include a sump for sediment storage, and a protected volume for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants. The minimum sediment & petroleum hydrocarbon storage capacity shall be as follows:

2.1.1	4 ft (1219 mm) Diameter OGS Units:	1.19 m^3 sediment / 265 L oil
	6 ft (1829 mm) Diameter OGS Units:	3.48 m^3 sediment / 609 L oil
	8 ft (2438 mm) Diameter OGS Units:	8.78 m^3 sediment / 1,071 L oil
	10 ft (3048 mm) Diameter OGS Units:	17.78 m^3 sediment / 1,673 L oil
	12 ft (3657 mm) Diameter OGS Units:	31.23 m^3 sediment / 2,476 L oil

PART 3 – PERFORMANCE & DESIGN

3.1 GENERAL

The OGS stormwater quality treatment device shall be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV). The OGS stormwater quality treatment device shall remove oil, sediment and gross pollutants from stormwater runoff during frequent wet weather events, and retain these pollutants during less frequent high flow wet weather events below the insert within the OGS for later removal during maintenance. The Manufacturer shall have at least ten (10) years of local experience, history and success in engineering design, manufacturing and production and supply of OGS stormwater quality treatment device systems, acceptable to the Engineer of Record.

3.2 SIZING METHODOLOGY

The OGS device shall be engineered, designed and sized to provide stormwater quality treatment based on treating a minimum of 90 percent of the average annual runoff volume and a minimum removal of an annual average 60% of the sediment (TSS) load based on the Particle Size Distribution (PSD) specified in the sizing report for the specified device. Sizing of the OGS shall be determined by use of a minimum ten (10) years of local historical rainfall data provided by Environment Canada. Sizing shall also be determined by use of the sediment removal performance data derived from the ISO 14034 ETV third-party verified laboratory testing data from testing conducted in accordance with the Canadian ETV protocol Procedure for Laboratory Testing of Oil-Grit Separators, as follows:

3.2.1 Sediment removal efficiency for a given surface loading rate and its associated flow rate shall be based on sediment removal efficiency demonstrated at the seven (7) tested surface loading rates specified in the protocol, ranging 40 L/min/m^2 to 1400 L/min/m^2, and as stated in the ISO 14034 ETV Verification Statement for the OGS device.

3.2.2 Sediment removal efficiency for surface loading rates between 40 L/min/m^2 and 1400 L/min/m^2 shall be based on linear interpolation of data between consecutive tested surface loading rates.

3.2.3 Sediment removal efficiency for surface loading rates less than the lowest tested surface loading rate of 40 L/min/m^2 shall be assumed to be identical to the sediment removal efficiency at 40 L/min/m^2. No extrapolation shall be allowed that results in a sediment removal efficiency that is greater than that demonstrated at 40 L/min/m^2.

3.2.4 Sediment removal efficiency for surface loading rates greater than the highest tested surface loading rate of 1400 L/min/m^2 shall assume zero sediment removal for the portion of flow that exceeds 1400 L/min/m^2, and shall be calculated using a simple proportioning formula, with 1400 L/min/m^2 in the numerator and the higher surface loading rate in the denominator, and multiplying the resulting fraction times the sediment removal efficiency at 1400 L/min/m^2.

The OGS device shall also have sufficient annual sediment storage capacity as specified and calculated in Section 2.1.

3.3 CANADIAN ETV or ISO 14034 ETV VERIFICATION OF SCOUR TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of third-party scour testing conducted in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**

3.3.1 To be acceptable for on-line installation, the OGS device must demonstrate an average scour test effluent concentration less than 10 mg/L at each surface loading rate tested, up to and including 2600 L/min/m^2.

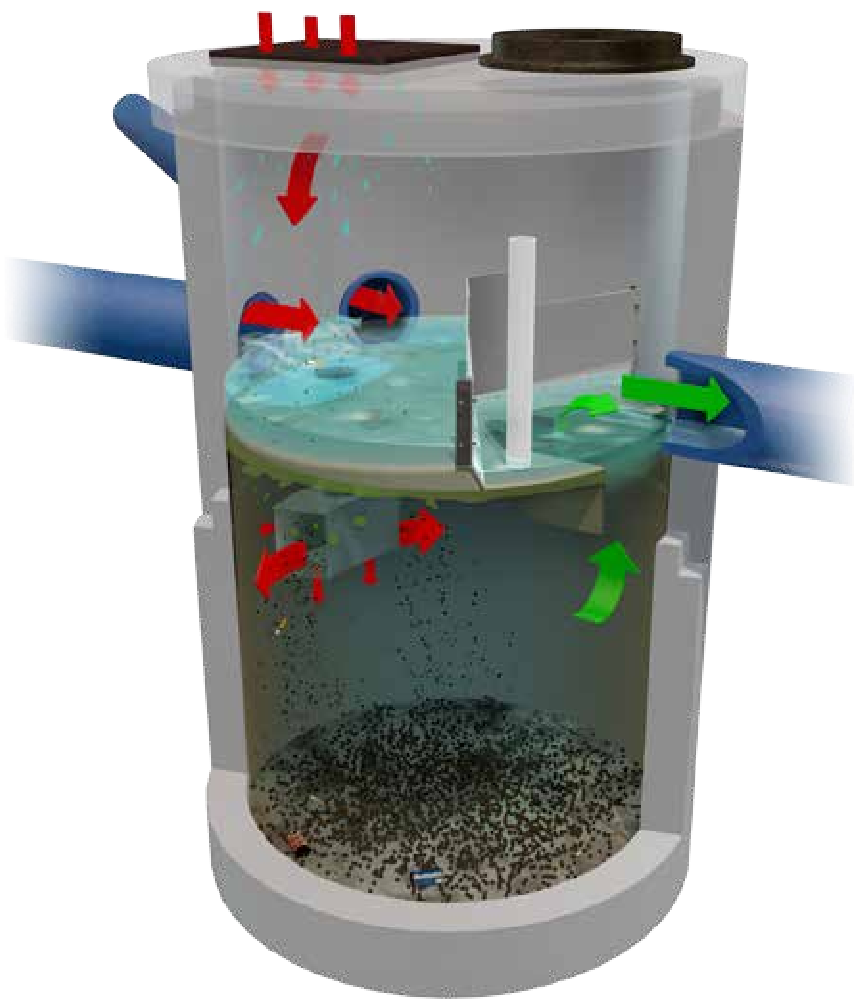
3.4 <u>LIGHT LIQUID RE-ENTRAINMENT SIMULATION TESTING</u>

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of completed third-party Light Liquid Re-entrainment Simulation Testing in accordance with the Canadian ETV **Program's Procedure for Laboratory Testing of Oil-Grit Separators,** with results reported within the Canadian ETV or ISO 14034 ETV verification. This re-entrainment testing is conducted with the device pre-loaded with low density polyethylene (LDPE) plastic beads as a surrogate for light liquids such as oil and fuel. Testing is conducted on the same OGS unit tested for sediment removal to assess whether light liquids captured after a spill are effectively retained at high flow rates.

3.4.1 For an OGS device to be an acceptable stormwater treatment device on a site where vehicular traffic occurs and the potential for an oil or fuel spill exists, the OGS device must have reported verified performance results of greater than 99% cumulative retention of LDPE plastic beads for the five specified surface loading rates (ranging 200 L/min/m^2 to 2600 L/min/m^2) in accordance with the Light Liquid Re-entrainment Simulation Testing within the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.** However, an OGS device shall not be allowed if the Light Liquid Re-entrainment Simulation Testing was performed with screening components within the OGS device that are effective at retaining the LDPE plastic beads, but would not be expected to retain light liquids such as oil and fuel.

Stormceptor® EF

Technical Manual



imbrium®

OVERVIEW

Stormceptor® EF is a continuation and evolution of the most globally recognized oil-grit separator (OGS) stormwater treatment technology - ***Stormceptor®***. Also known as a hydrodynamic separator, the enhanced flow Stormceptor EF is a high performing oil-grit separator that effectively removes a wide variety of pollutants from stormwater and snowmelt runoff at higher flow rates as compared to the original Stormceptor. Stormceptor EF captures and retains sediment (TSS), free oils, gross pollutants and other pollutants that attach to particles, such as nutrients and metals. Stormceptor EF's patent-pending treatment and scour prevention technology and internal bypass ensures sediment is retained during all rainfall events..

Stormceptor EF offers design flexibility in one simplified platform, accepting stormwater flow from a single inlet pipe, multiple inlet pipes, and/or from the surface through an inlet grate. Stormceptor EF can also serve as a junction structure, accommodate a 90-degree inlet to outlet bend angle, and be modified to ensure performance in submerged conditions. With its scour prevention technology and internal bypass, Stormceptor EF can be installed online, eliminating the need for costly additional bypass structures.

OPERATION

- Stormwater enters the Stormceptor upper chamber through the inlet pipe(s) or a surface inlet grate. A specially designed insert reduces the influent velocity by creating a pond upstream of the insert's weir. Sediment particles immediately begin to settle. Swirling flow sweeps water, sediment, and floatables across the sloped surface of the insert to the inlet opening of the drop pipe, where a strong vortex draws water, sediment, oil, and debris down the drop pipe cone.
- Influent exits the cone into the drop pipe duct. The duct has two large rectangular outlet openings as well as perforations in the backside and floor of the duct. Influent is diffused through these various opening in multiple directions and at low velocity into the lower chamber.
- Free oils and other floatables rise up and are trapped beneath the insert, while sediment settles to the sump. Pollutants are retained for later removal during maintenance cleaning.
- Treated effluent enters the outlet riser, moves upward, and discharges to the top side of the insert downstream of the weir, where it flows out the outlet pipe.
- During intense storm events with very high influent flow rates, the pond height on the upstream side of the weir may exceed the height of the weir, and the excess flow passes over the top of the weir to the downstream side of the insert, and exits through the outlet pipe. This internal bypass feature allows for online installation, avoiding the cost of additional bypass structures. During bypass, the pond separates sediment from all incoming flows, while full treatment in the lower chamber continues at the maximum flow rate.
- Stormceptor EF's patent-pending enhanced flow and scour prevention technology ensures pollutants are captured and retained, allowing excess flows to bypass during infrequent, high intensity storms.
- **Refer to components identified in Figures 1 and 2 to understand the Stormceptor EF operation.**

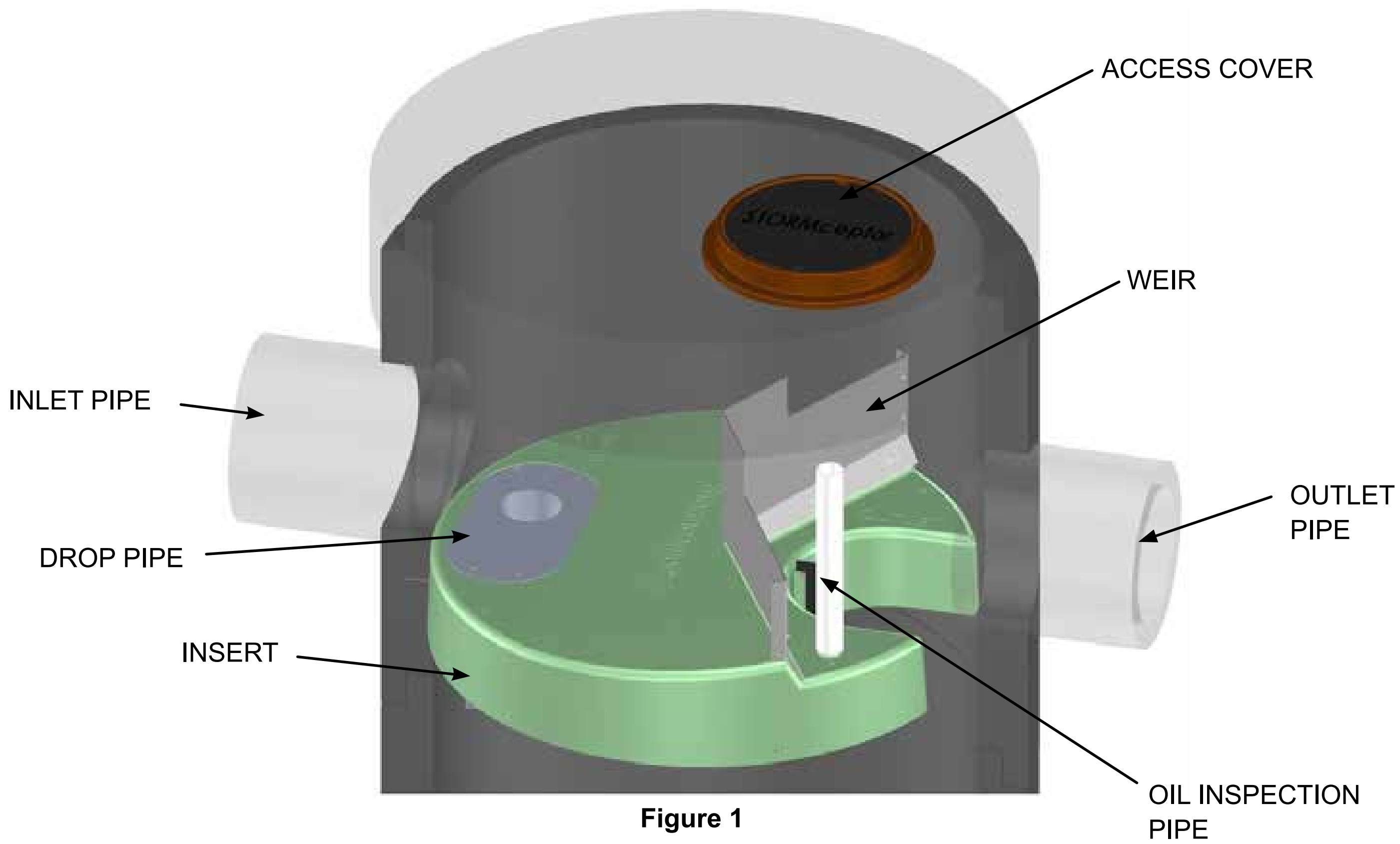


Figure 1

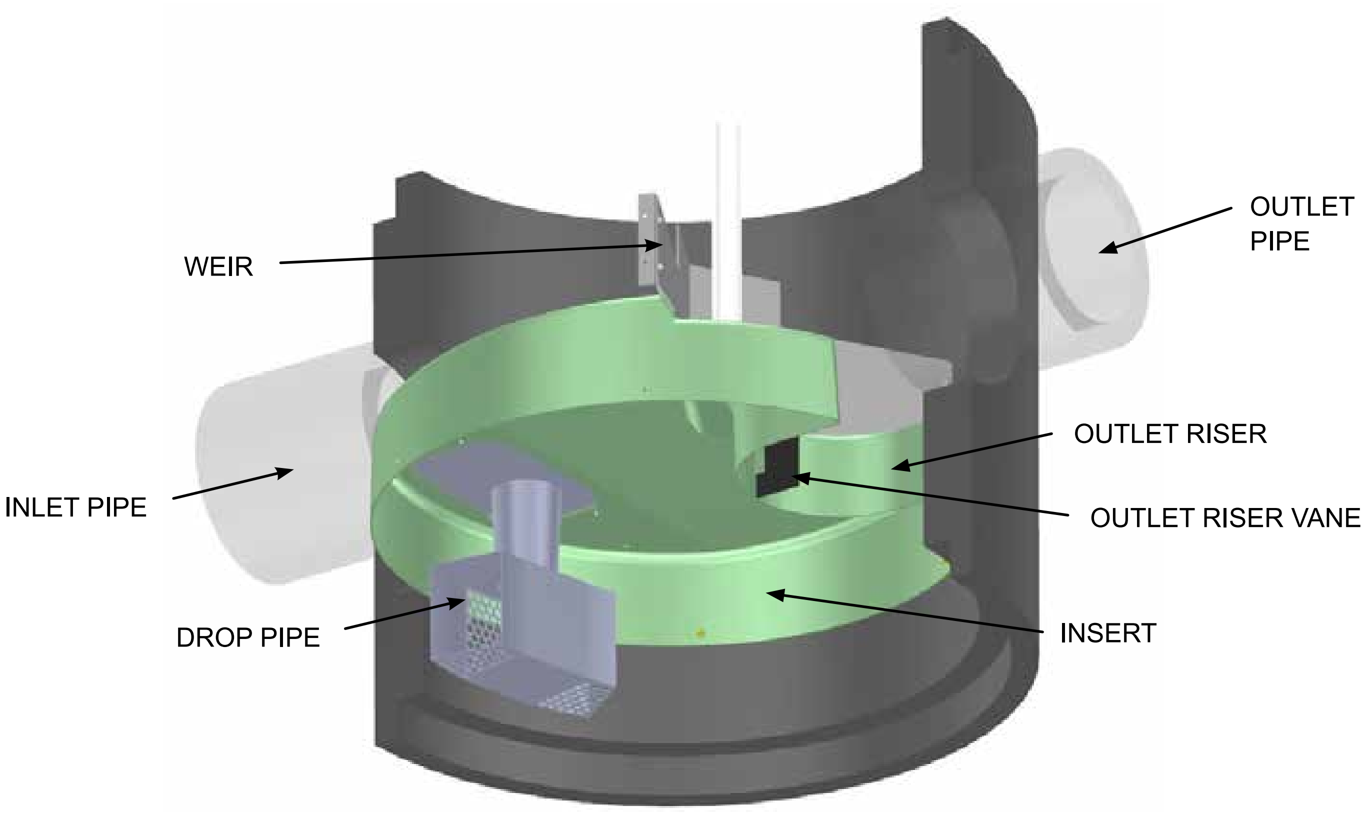


Figure 2

FEATURES AND BENEFITS

FEATURE	BENEFITS
Patent-pending enhanced flow, TSS treatment technology	Superior, verified third-party performance
Scour prevention with an internal bypass	Validated online installation and cost savings
Third-party verified light liquid capture (oil) and retention (Stormceptor EFO)	Proven performance for fuel/oil hotspot locations
Functions as bend, junction or inlet structure	Cost savings & design flexibility
Minimal drop between inlet and outlet	Site installation ease
Large diameter outlet riser for inspection and maintenance	Easy maintenance access from grade

APPLICATIONS

Stormceptor EF is designed as an 'at source' solution for commercial and industrial sites, urban environments, and residential developments. Stormceptor EF is ideal for:

- Pretreatment of wet ponds, filters, infiltration systems, bioretention, and other Low Impact Development (LID) applications
- Commercial sites
- Manufacturing/Industrial sites
- Residential developments
- Fueling stations, convenience stores, fast food restaurants
- Roads and highways
- Airports, seaports, and military bases
- Hydrocarbon spill, high pollutant load hotspots (Stormceptor EFO)

PRODUCT DETAILS

METRIC DIMENSIONS AND CAPACITIES								
Stormceptor Model	Inside Diameter	Minimum Surface to Outlet Invert Depth	Depth Below Outlet Pipe Invert	Wet Volume	Sediment Capacity[1]	Hydrocarbon Storage Capacity[2]	Maximum Flow Rate into Lower Chamber[3]	Peak Conveyance Flow Rate[4]
	(m)	(mm)	(mm)	(L)	(m^3)	(L)	(L/s)	(L/s)
EF4 / EFO4	1.22	915	1524	1780	1.19	265	22.1 / 10.4	425
EF6 / EFO6	1.83	915	1930	5070	3.47	610	49.6 / 23.4	990
EF8 / EFO8	2.44	1219	2591	12090	8.78	1070	88.3 / 41.6	1700
EF10 / EFO10	3.05	1219	3251	23700	17.79	1670	138 / 65	2830
EF12 / EFO12	3.66	1524	3886	40800	31.22	2475	198.7 / 93.7	2830

U.S. DIMENSIONS AND CAPACITIES								
Stormceptor Model	Inside Diameter	Minimum Surface to Outlet Invert Depth	Depth Below Outlet Pipe Invert	Wet Volume	Sediment Capacity[1]	Hydrocarbon Storage Capacity[2]	Maximum Flow Rate into Lower Chamber[3]	Peak Conveyance Flow Rate[4]
	(ft)	(in)	(in)	(gal)	(ft^3)	(gal)	(cfs)	(cfs)
EF4 / EFO4	4	36	60	471	42	70	0.78 / 0.37	15
EF6 / EFO6	6	36	76	1339	123	160	1.75 / 0.83	35
EF8 / EFO8	8	48	102	3194	310	280	3.12 / 1.47	60
EF10 / EFO10	10	48	128	6261	628	440	4.87 / 2.30	100
EF12 / EFO12	12	60	153	10779	1103	655	7.02 / 3.31	100

1. Sediment Capacity is measured from the floor to the bottom of the drop pipe cone. Sediment Capacity can be increased to accommodate specific site designs and pollutant loads. Contact your local representative for assistance.
2. Hydrocarbon Storage Capacity is measured from the bottom of the outlet riser to the underside of the insert. Hydrocarbon Storage Capacity can be increased to accommodate specific site designs and pollutant loads. Contact your local representative for assistance.
3. EF Maximum Flow Rate into Lower Chamber is based on a maximum surface loading rate (SLR) into the lower chamber of 1135 L/min/m^2 (27.9 gpm/ft^2). EFO Maximum Flow Rate into Lower Chamber is based on a maximum surface loading rate (SLR) into the lower chamber of 535 L/min/m^2 (13.1 gpm/ft^2).
4. Peak Conveyance Flow Rate is limited by a maximum velocity of 1.5 m/s (5 fps).

UNIT DESIGN

Sizing Methodology

Stormceptor® EF and Stormceptor® EFO are sized using local historical rainfall data for the site of interest, specific site parameters, and a performance curve for TSS removal derived from third-party testing conducted in accordance with the Canadian Environmental Technology Verification (ETV) Program's ***Procedure for Laboratory Testing of Oil-Grit Separators***. Every Stormceptor unit is designed to achieve the specified target TSS removal, however, for sites where oil/fuel capture and retention is an additional specified water quality objective Stormceptor EFO is the proper selection. The sizing methodology includes various considerations, including:

- Site parameters
- Local historical rainfall data
- Capture of the Canadian ETV particle size distribution
- Requirements for oil/fuel capture and retention
- Performance results from third-party testing and verification

State, provincial, and local regulatory agencies and municipalities may have specific sizing and design criteria for stormwater treatment systems such as OGS devices. To ensure proper sizing and design, contact your local Stormceptor representative for sizing and design assistance or visit www.imbriumsystems.com for more information.

ONLINE APPLICATION

Stormceptor EF's internal bypass and patent-pending scour prevention technology has demonstrated very effective retention of pollutants in third-party testing and verification following the Canadian ETV's ***Procedure for Laboratory Testing of Oil-Grit Separators***. Sediment scour prevention demonstrated an effluent concentration of less than 10 mg/L for sediment particles ranging from 1 to 1,000 microns, even during peak influent flow rates associated with infrequent high intensity storm events. While Stormceptor EF will capture oil, only the Stormceptor EFO configuration has been third-party tested and verified to retain greater than 99% of captured oil.

Based on these verified performance attributes, the most efficient and widely accepted application of Stormceptor EF is an online configuration, which allows all upstream conveyance flows to enter and exit the unit. The online application eliminates the need for costly additional bypass structures, piping and installation expense.

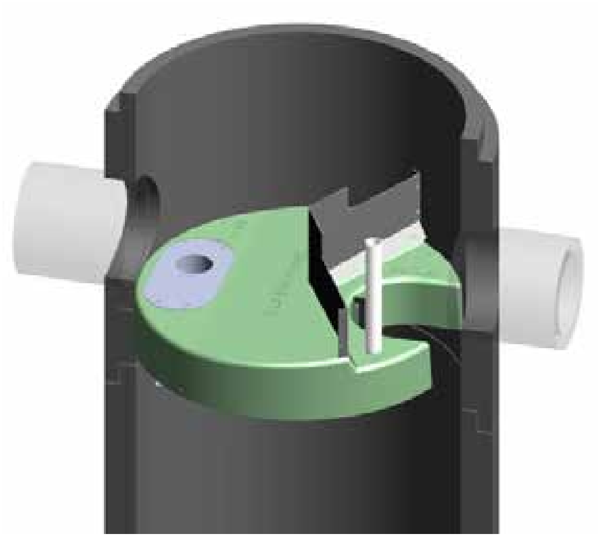


Figure 3

FLOW ENTRANCE OPTIONS

Single Inlet Pipe – A common design which includes one inlet pipe and one outlet pipe. A 90-degree (maximum) bend is also accepted with this configuration. **Example seen in Figure 3**.

MAXIMUM PIPE DIAMETER		
MODEL	INLET	OUTLET
	(in / mm)	(in / mm)
EF4 / EFO4	24 / 610	24 / 610
EF6 / EFO6	36 / 915	36 / 915
EF8 / EFO8	48 / 1220	48 / 1220
EF10 / EFO10	72 / 1828	72 / 1828
EF12 / EFO12	72 / 1828	72 / 1828

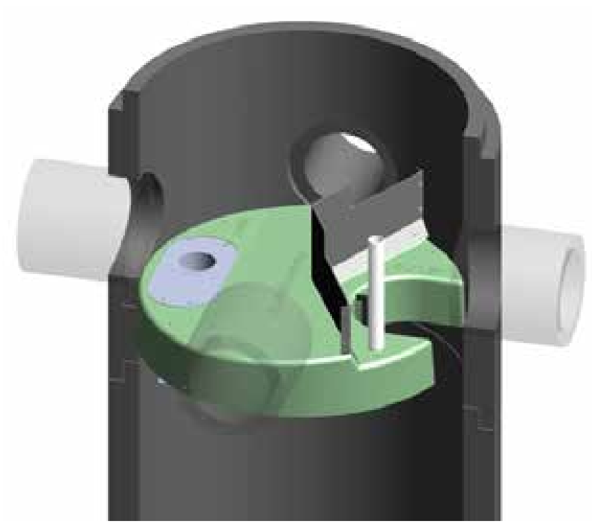


Figure 4

Multiple Inlet Pipes – Allows for multiple inlet pipes of various diameters to enter the unit. **Example seen in Figure 4**.

MAXIMUM PIPE DIAMETER		
MODEL	INLET	OUTLET
	(in / mm)	(in / mm)
EF4 / EFO4	18 / 457	24 / 610
EF6 / EFO6	30 / 762	36 / 915
EF8 / EFO8	42 / 1067	48 / 1220
EF10 / EFO10	60 / 1524	72 / 1828
EF12 / EFO12	60 / 1524	72 / 1828

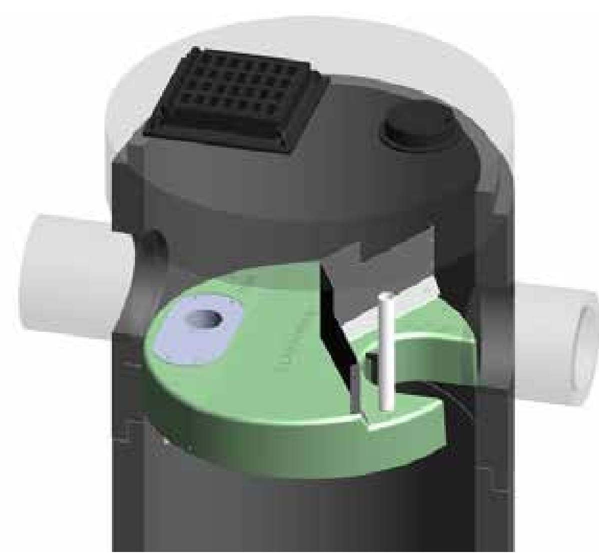


Figure 5

Inlet Grate – Allows surface runoff to enter the unit from grade. The inlet grate option can also be used in conjunction with one inlet pipe or multiple inlet pipes. A removable flow deflector is added in the Stormceptor EF4/EFO4. **Example seen in Figure 5**.

MAXIMUM PIPE DIAMETER		
MODEL	INLET	OUTLET
	(in / mm)	(in / mm)
EF4 / EFO4	24 / 610	24 / 610
EF6 / EFO6	36 / 915	36 / 915
EF8 / EFO8	48 / 1220	48 / 1220
EF10 / EFO10	72 / 1828	72 / 1828
EF12 / EFO12	72 / 1828	72 / 1828

INLET-TO-OUTLET DROP

Elevation differential between the inlet and outlet pipe inverts is dictated by the angle at which the inlet pipe(s) enters the unit (**illustration seen in Figure 6**).

0° – 45°: The inlet pipe is 1-inch (25mm) higher than the outlet pipe.

45° – 90°: The inlet pipe is 2-inches (50mm) higher than the outlet pipe.

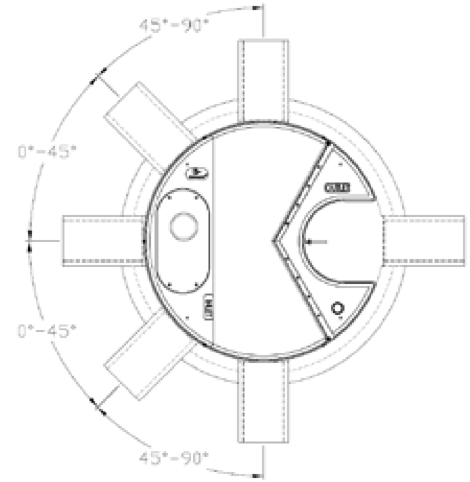


Figure 6

SUBMERGED (TAILWATER) DESIGN

Submerged or tailwater conditions are defined as standing water above the insert elevation during zero-runoff conditions. A weir height modification allows Stormceptor EF to operate under submerged conditions. The following information is necessary to properly design Stormceptor EF for the submerged condition:

- Stormceptor top of grade elevation
- Stormceptor outlet pipe invert elevation
- Standing water elevation

NOTE: The maximum weir height for Stormceptor EF is 48 inches (1200 mm). Contact your local Stormceptor representative for design assistance.

LIVE LOAD

Stormceptor EF is typically designed for local highway truck loading. In instances where other live loads are required, Stormceptor EF can be customized to meet the necessary structural requirements. Contact your local Stormceptor representative for design assistance.

SHALLOW COVER

Stormceptor EF is typically designed with a minimum depth of burial to the outlet invert based on the diameter of the inlet and outlet pipes. A common minimum burial depth to the outlet invert is 48 inches (1.2 meters). In instances where there may be site constraints to the depth of burial contact your local Stormceptor representative for design assistance.

HEAD LOSS

The head loss through Stormceptor EF is similar to that of a 60-degree bend structure. The applicable K value for calculating minor losses through the unit is 1.1. For submerged conditions the applicable K value is 3.0.

ABOVE-GROUND INSTALLATIONS

Stormceptor EF can be designed as a free-standing above-ground unit, constructed of fiberglass as illustrated in **Figure 7**. These customized units are lightweight and can be installed within a building footprint, providing structural support and installation advantages. Contact your local Stormceptor representative for design assistance.

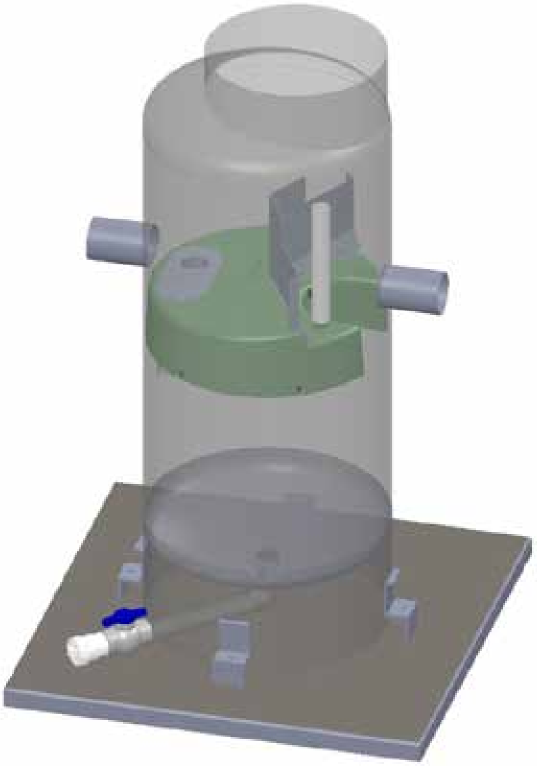


Figure 7

PERFORMANCE VERIFICATION TESTING

Stormceptor EF has been third-party performance tested according to the Canadian Environmental Technical Verification (ETV) Procedure for ***Laboratory Testing of Oil-Grit Separators***, and has received ISO 14034 Environmental Management – Environmental Technology Verification (ETV).

For more information, please visit www.imbriumsystems.com or contact your local Stormceptor representative.

INSTALLATION

For installation details, please visit www.imbriumsystems.com and refer to the Stormceptor EF Installation Guideline or contact your local Stormceptor representative.

INSPECTION AND MAINTENANCE

As with any stormwater treatment device, periodic inspection and maintenance of Stormceptor EF is required for long-term performance.

Inspection and maintenance is performed from grade without entering the unit. Sediment depth inspections are performed through the outlet riser, and oil presence can be determined through the oil inspection pipe. Oil presence and sediment depth are determined by inserting a Sludge Judge® or measuring stick to quantify the pollutant depths. Visual inspections of the insert can be performed to ensure there is no damage or blockages. A beneficial feature of Stormceptor EF in comparison to many other treatment practices is that once it is maintained, Stormceptor EF is functionally restored to its original condition.

When maintenance is required, a standard vacuum truck is used to remove the pollutants (sediment and floatables) from the lower chamber of the unit through the outlet riser. When an appreciable amount of oil or other hydrocarbons is present, these floatable pollutants can be removed by hydrovac from the water surface. Should an oil/fuel spill occur, or presence of oil/fuel be identified within the unit, it should be cleaned immediately by a licensed liquid waste hauler.

RECOMMENDED SEDIMENT DEPTHS FOR MAINTENANCE SERVICE*	
MODEL	Sediment Depth (in/mm)
EF4 / EFO4	8 / 203
EF6 / EFO6	12 /305
EF8 / EFO8	24 / 610
EF10 / EFO10	24 / 610
EF12 / EFO12	24 / 610

* Based on a minimum distance of 40 inches (1,016 mm) from bottom of outlet riser to top of sediment bed.

The frequency of inspection and maintenance may need to be adjusted based on site conditions to ensure the unit is operating and performing as intended. Maintenance costs will vary based on the size of the unit, site conditions, local requirements, location, and transportation distance(s).

For more details on inspection and maintenance refer to the Stormceptor EF Owner's Manual at www.imbriumsystems.com.

HYDROCARBON CAPTURE AND RETENTION

Stormceptor EFO
Stormceptor is often installed on high-traffic pollutant hotspots where hydrocarbon spill potential exists.

The technology platform of Stormceptor EFO is the same as Stormceptor EF, however the maximum surface loading rate into the lower chamber is restricted to a lower value with Stormceptor EFO, thereby ensuring excellent oil retention. Third-party testing in accordance with the Light Liquid Re-entrainment testing provisions within the Canadian ETV protocol ***Procedure for Laboratory Testing of Oil-Grit Separators*** demonstrated greater than 99% oil retention. Stormceptor EFO is engineered to capture and retain free floating oil/chemical/fuel spills, not emulsified hydrocarbons.

Optional Oil Alarm

Oil Sheen

When oil is present in stormwater runoff, a sheen may be noticeable at the Stormceptor outlet. An oil rainbow or sheen can be noticeable at very low oil concentrations (< 10 mg/L). Despite the appearance of a sheen, Stormceptor EFO may still be functioning as intended.

Disposal

Maintenance providers are to follow all federal, state/ provincial, and local requirements for disposal of hydrocarbons.

Oil Level Alarm

As an added safeguard, an oil level alarm is available as an optional feature for Stormceptor EFO. This is an electronic monitoring system designed to trigger a visual and audible alarm when a preset level of oil is captured in the lower chamber. The oil level alarm is installed as illustrated in **Figure 8**.

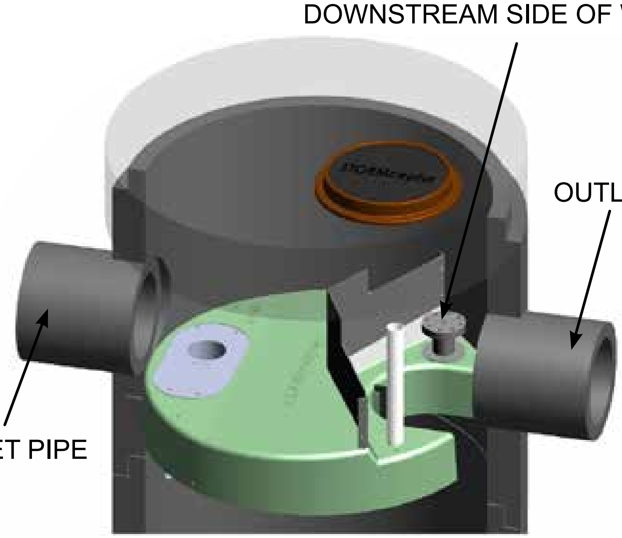


Figure 8

ADDITIONAL POLLUTANT STORAGE CAPACITY

Stormceptor EF/EFO can be easily modified to increase sediment storage capacity by extending the depth of the lower chamber. Stormceptor EFO can be modified to increase hydrocarbon storage capacity by extending the outlet riser, thereby providing the storage volumes depicted in the table below.

STORMCEPTOR EFO STORAGE VOLUME		
Stormceptor EFO Model	**Standard Hydrocarbon Storage Capacity [1]**	**Extended Hydrocarbon Storage Capacity [1,2]**
	(L / gal)	**(L / gal)**
EFO4	265 / 70	395 / 105
EFO6	610 / 160	1615 / 425
EFO8	1070 / 280	4340 / 1145
EFO10	1670 / 440	NA
EFO12	2475 / 655	NA

1. Hydrocarbon Storage Capacity is measured from the bottom of the outlet riser to the underside of the insert.
2. Distance from bottom of the extended outlet riser to top of the sediment maintenance depth is 914 mm (36 in).
   NA –Not available in these model sizes

Additional hydrocarbon storage capacity can be added with a draw off tank.

Contact your local Stormceptor representative for additional information and design assistance.

HEALTH AND SAFETY

For all aspects of installation and inspection/maintenance, OSHA and appropriate local regulations should be followed to ensure safe practice.

Contact

888-279-8826 / 416-960-9900

info@imbriumsystems.com

www.imbriumsystems.com

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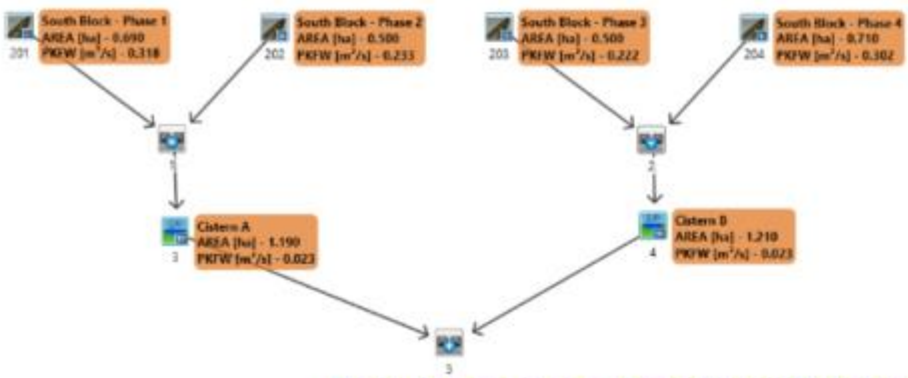
IM_STC_EF_11/17

APPENDIX

E

Hydrologic Modelling Results
(Visual OTTHYMO)



Post-development Conditions Schematic

```
********************************************
** SIMULATION:1 - 25mm DesignStormChicago **
********************************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A= 722.949
| Ptotal= 24.99 mm |                          B=   7.503
--------------------                          C=   0.862
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =  0.33

                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                 0.00    1.20 |  1.00   14.13 |  2.00    3.38 |  3.00    1.48
                 0.17    1.41 |  1.17   61.31 |  2.17    2.77 |  3.17    1.35
                 0.33    1.72 |  1.33   19.06 |  2.33    2.36 |  3.33    1.25
                 0.50    2.20 |  1.50    9.21 |  2.50    2.05 |  3.50    1.16
                 0.67    3.07 |  1.67    5.90 |  2.67    1.81 |  3.67    1.08
                 0.83    5.10 |  1.83    4.30 |  2.83    1.63 |  3.83    1.02


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0201)|   Area    (ha)=   0.69
|ID= 1 DT= 5.0 min |   Total Imp(%)=  88.00   Dir. Conn.(%)=  88.00
--------------------
                           IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.61        0.08
     Dep. Storage     (mm)=       1.00        1.50
     Average Slope     (%)=       1.00        2.00
     Length            (m)=      67.82       40.00
     Mannings n           =      0.013       0.250

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


                            ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    1.20 | 1.083   14.13 | 2.083    3.38 |  3.08    1.48
                0.167    1.20 | 1.167   14.13 | 2.167    3.38 |  3.17    1.48
                0.250    1.41 | 1.250   61.31 | 2.250    2.77 |  3.25    1.35
                0.333    1.41 | 1.333   61.31 | 2.333    2.77 |  3.33    1.35
                0.417    1.72 | 1.417   19.06 | 2.417    2.36 |  3.42    1.25
                0.500    1.72 | 1.500   19.06 | 2.500    2.36 |  3.50    1.25
                0.583    2.20 | 1.583    9.21 | 2.583    2.05 |  3.58    1.16
                0.667    2.20 | 1.667    9.21 | 2.667    2.05 |  3.67    1.16
```

```
                0.750    3.07 | 1.750    5.90 | 2.750    1.81 |  3.75    1.08
                0.833    3.07 | 1.833    5.90 | 2.833    1.81 |  3.83    1.08
                0.917    5.10 | 1.917    4.30 | 2.917    1.63 |  3.92    1.02
                1.000    5.10 | 2.000    4.30 | 3.000    1.63 |  4.00    1.02

     Max.Eff.Inten.(mm/hr)=      61.31        17.44
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       2.46 (ii)    6.33 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.30         0.15
                                                          *TOTALS*
     PEAK FLOW      (cms)=       0.10         0.00          0.105 (iii)
     TIME TO PEAK   (hrs)=       1.33         1.42           1.33
     RUNOFF VOLUME   (mm)=      23.99         8.08          22.08
     TOTAL RAINFALL  (mm)=      24.99        24.99          24.99
     RUNOFF COEFFICIENT  =       0.96         0.32           0.88

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.10) + QBASE(  0.55) =    0.65  (cms)

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0202)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                           IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.45        0.05
     Dep. Storage     (mm)=       1.00        1.50
     Average Slope     (%)=       1.00        2.00
     Length            (m)=      57.74       40.00
     Mannings n           =      0.013       0.250

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


                            ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    1.20 | 1.083   14.13 | 2.083    3.38 |  3.08    1.48
                0.167    1.20 | 1.167   14.13 | 2.167    3.38 |  3.17    1.48
                0.250    1.41 | 1.250   61.31 | 2.250    2.77 |  3.25    1.35
                0.333    1.41 | 1.333   61.31 | 2.333    2.77 |  3.33    1.35
                0.417    1.72 | 1.417   19.06 | 2.417    2.36 |  3.42    1.25
```

```
               0.500    1.72 | 1.500   19.06 | 2.500    2.36 |  3.50    1.25
               0.583    2.20 | 1.583    9.21 | 2.583    2.05 |  3.58    1.16
               0.667    2.20 | 1.667    9.21 | 2.667    2.05 |  3.67    1.16
               0.750    3.07 | 1.750    5.90 | 2.750    1.81 |  3.75    1.08
               0.833    3.07 | 1.833    5.90 | 2.833    1.81 |  3.83    1.08
               0.917    5.10 | 1.917    4.30 | 2.917    1.63 |  3.92    1.02
               1.000    5.10 | 2.000    4.30 | 3.000    1.63 |  4.00    1.02

     Max.Eff.Inten.(mm/hr)=      61.31        17.44
              over (min)        5.00        10.00
     Storage Coeff.  (min)=       2.23 (ii)    5.80 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.30         0.15
                                                        *TOTALS*
     PEAK FLOW       (cms)=       0.08         0.00          0.078 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.42           1.33
     RUNOFF VOLUME    (mm)=      23.99         8.08          22.40
     TOTAL RAINFALL   (mm)=      24.99        24.99          24.99
     RUNOFF COEFFICIENT   =       0.96         0.32           0.90

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

--------------------
| ADD HYD  (  0001)|
|   1 +  2 =  3    |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0201):    0.69   0.105     1.33    22.08
      + ID2= 2 (  0202):    0.50   0.078     1.33    22.40
        ====================================================
        ID = 3 (  0001):    1.19   0.182     1.33    22.21

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.182) + BASE (  0.550) =   0.732  (cms).
--------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0003)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0610     0.1006
                          0.0010     0.0488   |   0.0850     0.1769
                          0.0240     0.0732   |   0.0000     0.0000
```

```
                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
  INFLOW : ID= 2 (  0001)      1.190      0.182      1.33      22.21
  OUTFLOW: ID= 1 (  0003)      1.190      0.001      4.08      11.77

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  0.29
                  TIME SHIFT OF PEAK FLOW        (min)=165.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=  0.0259

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0203)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  80.00   Dir. Conn.(%)=  80.00
--------------------
                               IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.40         0.10
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n          =      0.013        0.250

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


                               ---- TRANSFORMED HYETOGRAPH ----
                   TIME    RAIN |  TIME    RAIN |'  TIME     RAIN |  TIME    RAIN
                    hrs   mm/hr |   hrs   mm/hr |'   hrs    mm/hr |   hrs   mm/hr
                  0.083    1.20 | 1.083   14.13 | 2.083    3.38 |  3.08    1.48
                  0.167    1.20 | 1.167   14.13 | 2.167    3.38 |  3.17    1.48
                  0.250    1.41 | 1.250   61.31 | 2.250    2.77 |  3.25    1.35
                  0.333    1.41 | 1.333   61.31 | 2.333    2.77 |  3.33    1.35
                  0.417    1.72 | 1.417   19.06 | 2.417    2.36 |  3.42    1.25
                  0.500    1.72 | 1.500   19.06 | 2.500    2.36 |  3.50    1.25
                  0.583    2.20 | 1.583    9.21 | 2.583    2.05 |  3.58    1.16
                  0.667    2.20 | 1.667    9.21 | 2.667    2.05 |  3.67    1.16
                  0.750    3.07 | 1.750    5.90 | 2.750    1.81 |  3.75    1.08
                  0.833    3.07 | 1.833    5.90 | 2.833    1.81 |  3.83    1.08
                  0.917    5.10 | 1.917    4.30 | 2.917    1.63 |  3.92    1.02
                  1.000    5.10 | 2.000    4.30 | 3.000    1.63 |  4.00    1.02

     Max.Eff.Inten.(mm/hr)=      61.31        17.44
              over (min)        5.00        10.00
     Storage Coeff.  (min)=       2.23 (ii)    7.16 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.30         0.14
                                                        *TOTALS*
     PEAK FLOW       (cms)=       0.07         0.00          0.070 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.42           1.33
```

```
     RUNOFF VOLUME    (mm)=      23.99         8.08          20.80
     TOTAL RAINFALL   (mm)=      24.99        24.99          24.99
     RUNOFF COEFFICIENT   =       0.96         0.32           0.83

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.07) + QBASE(  0.55) =    0.62  (cms)

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD (  0204)|   Area    (ha)=   0.71
|ID= 1 DT= 5.0 min |   Total Imp(%)=  78.00   Dir. Conn.(%)=  78.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.55         0.16
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      68.80        40.00
     Mannings n           =      0.013        0.250

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


                               ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    1.20 | 1.083   14.13 | 2.083    3.38 |  3.08    1.48
                0.167    1.20 | 1.167   14.13 | 2.167    3.38 |  3.17    1.48
                0.250    1.41 | 1.250   61.31 | 2.250    2.77 |  3.25    1.35
                0.333    1.41 | 1.333   61.31 | 2.333    2.77 |  3.33    1.35
                0.417    1.72 | 1.417   19.06 | 2.417    2.36 |  3.42    1.25
                0.500    1.72 | 1.500   19.06 | 2.500    2.36 |  3.50    1.25
                0.583    2.20 | 1.583    9.21 | 2.583    2.05 |  3.58    1.16
                0.667    2.20 | 1.667    9.21 | 2.667    2.05 |  3.67    1.16
                0.750    3.07 | 1.750    5.90 | 2.750    1.81 |  3.75    1.08
                0.833    3.07 | 1.833    5.90 | 2.833    1.81 |  3.83    1.08
                0.917    5.10 | 1.917    4.30 | 2.917    1.63 |  3.92    1.02
                1.000    5.10 | 2.000    4.30 | 3.000    1.63 |  4.00    1.02

     Max.Eff.Inten.(mm/hr)=      61.31        12.92
                over (min)        5.00        20.00
     Storage Coeff.  (min)=       2.48 (ii)   18.48 (ii)
     Unit Hyd. Tpeak (min)=       5.00        20.00
     Unit Hyd. peak  (cms)=       0.29         0.06
```

```
                                                                  *TOTALS*
     PEAK FLOW       (cms)=       0.09         0.00           0.094 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.58            1.33
     RUNOFF VOLUME    (mm)=      23.99         8.08           20.47
     TOTAL RAINFALL   (mm)=      24.99        24.99           24.99
     RUNOFF COEFFICIENT   =       0.96         0.32            0.82

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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


--------------------
| ADD HYD  (  0002)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0203):     0.50   0.070     1.33    20.80
      + ID2= 2 (  0204):     0.71   0.094     1.33    20.47
        ===================================================
        ID = 3 (  0002):     1.21   0.164     1.33    20.61

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.164) + BASE (  0.550) =   0.714  (cms).
--------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0590     0.0998
                          0.0010     0.0484   |   0.0830     0.1754
                          0.0240     0.0726   |   0.0000     0.0000

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0002)      1.210      0.164      1.33      20.61
   OUTFLOW: ID= 1 (  0004)      1.210      0.001      4.08      10.42

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  0.31
                   TIME SHIFT OF PEAK FLOW        (min)=165.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0245

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```
| ADD HYD  (  0005)|
|   1 +  2 =  3    |        AREA    QPEAK    TPEAK     R.V.
--------------------        (ha)    (cms)    (hrs)     (mm)
       ID1= 1 (  0003):     1.19   0.001     4.08    11.77
     + ID2= 2 (  0004):     1.21   0.001     4.08    10.42
       ==================================================
       ID = 3 (  0005):     2.40   0.001     4.08    11.09

    NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
    ACTUAL PEAK FLOW: PEAK (  0.001) + BASE (  1.100) =   1.101  (cms).
--------------------------------------------------------------------------------
*******************************************
** SIMULATION:2 - 25yr 4hr 10min Chicago **
*******************************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1234.000
| Ptotal= 65.15 mm |                          B=   5.500
--------------------                          C=   0.786
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =  0.33

                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                 0.00    4.31 |  1.00   34.08 |  2.00   10.19 |  3.00    5.13
                 0.17    4.94 |  1.17  143.01 |  2.17    8.67 |  3.17    4.76
                 0.33    5.82 |  1.33   44.65 |  2.33    7.57 |  3.33    4.45
                 0.50    7.15 |  1.50   23.56 |  2.50    6.74 |  3.50    4.18
                 0.67    9.42 |  1.67   16.20 |  2.67    6.09 |  3.67    3.95
                 0.83   14.31 |  1.83   12.46 |  2.83    5.56 |  3.83    3.74


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0201)|   Area    (ha)=   0.69
|ID= 1 DT= 5.0 min |   Total Imp(%)=  88.00   Dir. Conn.(%)=  88.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
    Surface Area     (ha)=       0.61         0.08
    Dep. Storage     (mm)=       1.00         1.50
    Average Slope     (%)=       1.00         2.00
    Length            (m)=      67.82        40.00
    Mannings n           =      0.013        0.250

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

```
                             ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    4.31 | 1.083   34.08 | 2.083   10.19 |  3.08    5.13
                0.167    4.31 | 1.167   34.08 | 2.167   10.19 |  3.17    5.13
                0.250    4.94 | 1.250  143.01 | 2.250    8.67 |  3.25    4.76
                0.333    4.94 | 1.333  143.01 | 2.333    8.67 |  3.33    4.76
                0.417    5.82 | 1.417   44.65 | 2.417    7.57 |  3.42    4.45
                0.500    5.82 | 1.500   44.65 | 2.500    7.57 |  3.50    4.45
                0.583    7.15 | 1.583   23.56 | 2.583    6.74 |  3.58    4.18
                0.667    7.15 | 1.667   23.56 | 2.667    6.74 |  3.67    4.18
                0.750    9.42 | 1.750   16.20 | 2.750    6.09 |  3.75    3.95
                0.833    9.42 | 1.833   16.20 | 2.833    6.09 |  3.83    3.95
                0.917   14.31 | 1.917   12.46 | 2.917    5.56 |  3.92    3.74
                1.000   14.31 | 2.000   12.46 | 3.000    5.56 |  4.00    3.74

     Max.Eff.Inten.(mm/hr)=     143.01        80.01
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.75 (ii)    4.51 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=       0.32         0.23
                                                          *TOTALS*
     PEAK FLOW       (cms)=       0.24         0.02         0.258 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33          1.33
     RUNOFF VOLUME    (mm)=      64.15        37.35         60.93
     TOTAL RAINFALL   (mm)=      65.15        65.15         65.15
     RUNOFF COEFFICIENT   =       0.98         0.57          0.94

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.26) + QBASE(  0.55) =    0.81  (cms)

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0202)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.45         0.05
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n           =      0.013        0.250
```

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


                              ---- TRANSFORMED HYETOGRAPH ----
                   TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                    hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                  0.083    4.31 | 1.083   34.08 | 2.083   10.19 |  3.08    5.13
                  0.167    4.31 | 1.167   34.08 | 2.167   10.19 |  3.17    5.13
                  0.250    4.94 | 1.250  143.01 | 2.250    8.67 |  3.25    4.76
                  0.333    4.94 | 1.333  143.01 | 2.333    8.67 |  3.33    4.76
                  0.417    5.82 | 1.417   44.65 | 2.417    7.57 |  3.42    4.45
                  0.500    5.82 | 1.500   44.65 | 2.500    7.57 |  3.50    4.45
                  0.583    7.15 | 1.583   23.56 | 2.583    6.74 |  3.58    4.18
                  0.667    7.15 | 1.667   23.56 | 2.667    6.74 |  3.67    4.18
                  0.750    9.42 | 1.750   16.20 | 2.750    6.09 |  3.75    3.95
                  0.833    9.42 | 1.833   16.20 | 2.833    6.09 |  3.83    3.95
                  0.917   14.31 | 1.917   12.46 | 2.917    5.56 |  3.92    3.74
                  1.000   14.31 | 2.000   12.46 | 3.000    5.56 |  4.00    3.74

      Max.Eff.Inten.(mm/hr)=     143.01        80.01
                 over (min)       5.00         5.00
      Storage Coeff.  (min)=       1.59 (ii)    4.13 (ii)
      Unit Hyd. Tpeak (min)=       5.00         5.00
      Unit Hyd. peak  (cms)=       0.33         0.24
                                                             *TOTALS*
      PEAK FLOW      (cms)=        0.18         0.01           0.190 (iii)
      TIME TO PEAK   (hrs)=        1.33         1.33            1.33
      RUNOFF VOLUME    (mm)=      64.15        37.35           61.47
      TOTAL RAINFALL   (mm)=      65.15        65.15           65.15
      RUNOFF COEFFICIENT   =       0.98         0.57            0.94

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

        (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
             CN*  =  85.0    Ia = Dep. Storage  (Above)
       (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
            THAN THE STORAGE COEFFICIENT.
      (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

 --------------------
 | ADD HYD  (  0001)|
 |   1 +  2 =  3    |          AREA    QPEAK    TPEAK     R.V.
 --------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 1 (  0201):      0.69    0.258     1.33    60.93
       + ID2= 2 (  0202):      0.50    0.190     1.33    61.47
         ==================================================
         ID = 3 (  0001):      1.19    0.448     1.33    61.16
```

```
     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.448) + BASE (  0.550) =   0.998  (cms).
 --------------------------------------------------------------------------------

 --------------------
 | RESERVOIR(  0003)|     OVERFLOW IS OFF
 | IN= 2---> OUT= 1 |
 | DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
 --------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000    |  0.0610     0.1006
                          0.0010     0.0488    |  0.0850     0.1769
                          0.0240     0.0732    |  0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0001)      1.190      0.448      1.33      61.16
   OUTFLOW: ID= 1 (  0003)      1.190      0.014      3.58      48.07

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  3.09
                   TIME SHIFT OF PEAK FLOW        (min)=135.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0624

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

 --------------------
 | CALIB            |
 | STANDHYD (  0203)|   Area    (ha)=   0.50
 |ID= 1 DT= 5.0 min |   Total Imp(%)=  80.00   Dir. Conn.(%)=  80.00
 --------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.40         0.10
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n           =      0.013        0.250

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


                              ---- TRANSFORMED HYETOGRAPH ----
                   TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                    hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                  0.083    4.31 | 1.083   34.08 | 2.083   10.19 |  3.08    5.13
                  0.167    4.31 | 1.167   34.08 | 2.167   10.19 |  3.17    5.13
                  0.250    4.94 | 1.250  143.01 | 2.250    8.67 |  3.25    4.76
                  0.333    4.94 | 1.333  143.01 | 2.333    8.67 |  3.33    4.76
                  0.417    5.82 | 1.417   44.65 | 2.417    7.57 |  3.42    4.45
                  0.500    5.82 | 1.500   44.65 | 2.500    7.57 |  3.50    4.45
                  0.583    7.15 | 1.583   23.56 | 2.583    6.74 |  3.58    4.18
                  0.667    7.15 | 1.667   23.56 | 2.667    6.74 |  3.67    4.18
                  0.750    9.42 | 1.750   16.20 | 2.750    6.09 |  3.75    3.95
                  0.833    9.42 | 1.833   16.20 | 2.833    6.09 |  3.83    3.95
```

```
             0.917   14.31 | 1.917   12.46 | 2.917    5.56 |  3.92    3.74
             1.000   14.31 | 2.000   12.46 | 3.000    5.56 |  4.00    3.74

     Max.Eff.Inten.(mm/hr)=     143.01        80.01
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.59 (ii)    5.11 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.33         0.16
                                                         *TOTALS*
     PEAK FLOW      (cms)=        0.16         0.02          0.175 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.42           1.33
     RUNOFF VOLUME    (mm)=      64.15        37.35          58.78
     TOTAL RAINFALL   (mm)=      65.15        65.15          65.15
     RUNOFF COEFFICIENT   =       0.98         0.57           0.90

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.17) + QBASE(  0.55) =    0.72  (cms)

-------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD (  0204)|   Area    (ha)=   0.71
|ID= 1 DT= 5.0 min |   Total Imp(%)=  78.00   Dir. Conn.(%)=  78.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.55         0.16
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      68.80        40.00
     Mannings n          =      0.013        0.250

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


                               ---- TRANSFORMED HYETOGRAPH ----
              TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
               hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
             0.083    4.31 | 1.083   34.08 | 2.083   10.19 |  3.08    5.13
             0.167    4.31 | 1.167   34.08 | 2.167   10.19 |  3.17    5.13
             0.250    4.94 | 1.250  143.01 | 2.250    8.67 |  3.25    4.76
             0.333    4.94 | 1.333  143.01 | 2.333    8.67 |  3.33    4.76
             0.417    5.82 | 1.417   44.65 | 2.417    7.57 |  3.42    4.45
             0.500    5.82 | 1.500   44.65 | 2.500    7.57 |  3.50    4.45
             0.583    7.15 | 1.583   23.56 | 2.583    6.74 |  3.58    4.18
```

```
             0.667    7.15 | 1.667   23.56 | 2.667    6.74 |  3.67    4.18
             0.750    9.42 | 1.750   16.20 | 2.750    6.09 |  3.75    3.95
             0.833    9.42 | 1.833   16.20 | 2.833    6.09 |  3.83    3.95
             0.917   14.31 | 1.917   12.46 | 2.917    5.56 |  3.92    3.74
             1.000   14.31 | 2.000   12.46 | 3.000    5.56 |  4.00    3.74

     Max.Eff.Inten.(mm/hr)=     143.01        80.01
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.77 (ii)    5.46 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.32         0.16
                                                         *TOTALS*
     PEAK FLOW      (cms)=        0.22         0.03          0.244 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.42           1.33
     RUNOFF VOLUME    (mm)=      64.15        37.35          58.25
     TOTAL RAINFALL   (mm)=      65.15        65.15          65.15
     RUNOFF COEFFICIENT   =       0.98         0.57           0.89

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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


--------------------
| ADD HYD  (  0002)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0203):     0.50   0.175     1.33    58.78
      + ID2= 2 (  0204):     0.71   0.244     1.33    58.25
        ===============================================
        ID = 3 (  0002):     1.21   0.418     1.33    58.47

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.418) + BASE (  0.550) =   0.968  (cms).
-------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0590     0.0998
                          0.0010     0.0484   |   0.0830     0.1754
                          0.0240     0.0726   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
```

```
                              (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0002)      1.210      0.418      1.33      58.47
   OUTFLOW: ID= 1 (  0004)      1.210      0.013      3.75      45.85

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  3.14
                  TIME SHIFT OF PEAK FLOW         (min)=145.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=  0.0612

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

--------------------
| ADD HYD  (  0005)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0003):     1.19    0.014     3.58    48.07
      + ID2= 2 (  0004):     1.21    0.013     3.75    45.85
        ===================================================
        ID = 3 (  0005):     2.40    0.027     3.67    46.95

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.027) + BASE (  1.100) =   1.127  (cms).
--------------------------------------------------------------------------------
********************************************
** SIMULATION:3 - 100yr 4hr 10min Chicago **
********************************************
--------------------
| CHICAGO STORM    |    IDF curve parameters: A=1435.000
| Ptotal= 80.66 mm |                          B=   5.200
--------------------                          C=   0.775
                        used in:   INTENSITY =  A / (t + B)^C

                        Duration of storm  =  4.00 hrs
                        Storm time step    = 10.00 min
                        Time to peak ratio =  0.33

                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                 0.00    5.55 |  1.00   41.69 |  2.00   12.86 |  3.00    6.58
                 0.17    6.34 |  1.17  174.10 |  2.17   10.98 |  3.17    6.12
                 0.33    7.44 |  1.33   54.37 |  2.33    9.62 |  3.33    5.73
                 0.50    9.10 |  1.50   29.07 |  2.50    8.59 |  3.50    5.39
                 0.67   11.90 |  1.67   20.18 |  2.67    7.78 |  3.67    5.09
                 0.83   17.89 |  1.83   15.63 |  2.83    7.12 |  3.83    4.83


--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0201)|   Area    (ha)=   0.69
|ID= 1 DT= 5.0 min |   Total Imp(%)=  88.00   Dir. Conn.(%)=  88.00
```

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--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.61         0.08
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      67.82        40.00
     Mannings n          =      0.013        0.250

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


                            ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    5.55 | 1.083   41.69 | 2.083   12.86 |  3.08    6.58
                0.167    5.55 | 1.167   41.69 | 2.167   12.86 |  3.17    6.58
                0.250    6.34 | 1.250  174.10 | 2.250   10.98 |  3.25    6.12
                0.333    6.34 | 1.333  174.10 | 2.333   10.98 |  3.33    6.12
                0.417    7.44 | 1.417   54.37 | 2.417    9.62 |  3.42    5.73
                0.500    7.44 | 1.500   54.37 | 2.500    9.62 |  3.50    5.73
                0.583    9.10 | 1.583   29.07 | 2.583    8.59 |  3.58    5.39
                0.667    9.10 | 1.667   29.07 | 2.667    8.59 |  3.67    5.39
                0.750   11.90 | 1.750   20.18 | 2.750    7.78 |  3.75    5.09
                0.833   11.90 | 1.833   20.18 | 2.833    7.78 |  3.83    5.09
                0.917   17.89 | 1.917   15.63 | 2.917    7.12 |  3.92    4.83
                1.000   17.89 | 2.000   15.63 | 3.000    7.12 |  4.00    4.83

     Max.Eff.Inten.(mm/hr)=     174.10       108.56
                over (min)       5.00         5.00
     Storage Coeff.  (min)=       1.62 (ii)    4.17 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=       0.32         0.24
                                                           *TOTALS*
     PEAK FLOW      (cms)=       0.29         0.02          0.318 (iii)
     TIME TO PEAK   (hrs)=       1.33         1.33           1.33
     RUNOFF VOLUME   (mm)=      79.66        50.54          76.16
     TOTAL RAINFALL  (mm)=      80.66        80.66          80.66
     RUNOFF COEFFICIENT  =       0.99         0.63           0.94

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.32) + QBASE(  0.55) =    0.87  (cms)

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| CALIB              |
| STANDHYD (  0202)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.45         0.05
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n           =      0.013        0.250

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


                               ---- TRANSFORMED HYETOGRAPH ----
                 TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                0.083    5.55 | 1.083   41.69 | 2.083   12.86 |  3.08    6.58
                0.167    5.55 | 1.167   41.69 | 2.167   12.86 |  3.17    6.58
                0.250    6.34 | 1.250  174.10 | 2.250   10.98 |  3.25    6.12
                0.333    6.34 | 1.333  174.10 | 2.333   10.98 |  3.33    6.12
                0.417    7.44 | 1.417   54.37 | 2.417    9.62 |  3.42    5.73
                0.500    7.44 | 1.500   54.37 | 2.500    9.62 |  3.50    5.73
                0.583    9.10 | 1.583   29.07 | 2.583    8.59 |  3.58    5.39
                0.667    9.10 | 1.667   29.07 | 2.667    8.59 |  3.67    5.39
                0.750   11.90 | 1.750   20.18 | 2.750    7.78 |  3.75    5.09
                0.833   11.90 | 1.833   20.18 | 2.833    7.78 |  3.83    5.09
                0.917   17.89 | 1.917   15.63 | 2.917    7.12 |  3.92    4.83
                1.000   17.89 | 2.000   15.63 | 3.000    7.12 |  4.00    4.83

     Max.Eff.Inten.(mm/hr)=     174.10       108.56
                over (min)       5.00         5.00
     Storage Coeff.  (min)=       1.47 (ii)    3.82 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=       0.33         0.25
                                                           *TOTALS*
     PEAK FLOW       (cms)=       0.22         0.02          0.233 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33           1.33
     RUNOFF VOLUME    (mm)=      79.66        50.54          76.74
     TOTAL RAINFALL   (mm)=      80.66        80.66          80.66
     RUNOFF COEFFICIENT   =       0.99         0.63           0.95

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
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--------------------
| ADD HYD  (  0001)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0201):     0.69   0.318     1.33    76.16
      + ID2= 2 (  0202):     0.50   0.233     1.33    76.74
        ================================================
        ID = 3 (  0001):     1.19   0.550     1.33    76.41

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.550) + BASE (  0.550) =   1.100  (cms).
--------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0003)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0610     0.1006
                          0.0010     0.0488   |   0.0850     0.1769
                          0.0240     0.0732   |   0.0000     0.0000

                                 AREA     QPEAK     TPEAK       R.V.
                                 (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0001)      1.190      0.550      1.33      76.41
   OUTFLOW: ID= 1 (  0003)      1.190      0.023      2.92      63.19

                  PEAK   FLOW   REDUCTION [Qout/Qin](%)=  4.26
                  TIME SHIFT OF PEAK FLOW        (min)= 95.00
                  MAXIMUM  STORAGE   USED       (ha.m.)=  0.0726

--------------------------------------------------------------------------------
--------------------
| CALIB              |
| STANDHYD (  0203)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  80.00   Dir. Conn.(%)=  80.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.40         0.10
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n           =      0.013        0.250

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


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

```
                   hrs    mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs    mm/hr
                 0.083     5.55 | 1.083   41.69 | 2.083   12.86 |  3.08    6.58
                 0.167     5.55 | 1.167   41.69 | 2.167   12.86 |  3.17    6.58
                 0.250     6.34 | 1.250  174.10 | 2.250   10.98 |  3.25    6.12
                 0.333     6.34 | 1.333  174.10 | 2.333   10.98 |  3.33    6.12
                 0.417     7.44 | 1.417   54.37 | 2.417    9.62 |  3.42    5.73
                 0.500     7.44 | 1.500   54.37 | 2.500    9.62 |  3.50    5.73
                 0.583     9.10 | 1.583   29.07 | 2.583    8.59 |  3.58    5.39
                 0.667     9.10 | 1.667   29.07 | 2.667    8.59 |  3.67    5.39
                 0.750    11.90 | 1.750   20.18 | 2.750    7.78 |  3.75    5.09
                 0.833    11.90 | 1.833   20.18 | 2.833    7.78 |  3.83    5.09
                 0.917    17.89 | 1.917   15.63 | 2.917    7.12 |  3.92    4.83
                 1.000    17.89 | 2.000   15.63 | 3.000    7.12 |  4.00    4.83

     Max.Eff.Inten.(mm/hr)=     174.10       108.56
                over (min)        5.00         5.00
     Storage Coeff.  (min)=       1.47 (ii)    4.72 (ii)
     Unit Hyd. Tpeak (min)=       5.00         5.00
     Unit Hyd. peak  (cms)=       0.33         0.22
                                                        *TOTALS*
     PEAK FLOW       (cms)=       0.19         0.03       0.222 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.33        1.33
     RUNOFF VOLUME    (mm)=      79.66        50.54       73.83
     TOTAL RAINFALL   (mm)=      80.66        80.66       80.66
     RUNOFF COEFFICIENT   =       0.99         0.63        0.92

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.22) + QBASE(  0.55) =    0.77  (cms)

--------------------------------------------------------------------------------
--------------------
| CALIB             |
| STANDHYD (  0204)|   Area    (ha)=   0.71
|ID= 1 DT= 5.0 min |   Total Imp(%)=  78.00   Dir. Conn.(%)=  78.00
--------------------
                               IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.55         0.16
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      68.80        40.00
     Mannings n           =      0.013        0.250

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

```
                                 ---- TRANSFORMED HYETOGRAPH ----
                    TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                     hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                   0.083    5.55 | 1.083   41.69 | 2.083   12.86 |  3.08    6.58
                   0.167    5.55 | 1.167   41.69 | 2.167   12.86 |  3.17    6.58
                   0.250    6.34 | 1.250  174.10 | 2.250   10.98 |  3.25    6.12
                   0.333    6.34 | 1.333  174.10 | 2.333   10.98 |  3.33    6.12
                   0.417    7.44 | 1.417   54.37 | 2.417    9.62 |  3.42    5.73
                   0.500    7.44 | 1.500   54.37 | 2.500    9.62 |  3.50    5.73
                   0.583    9.10 | 1.583   29.07 | 2.583    8.59 |  3.58    5.39
                   0.667    9.10 | 1.667   29.07 | 2.667    8.59 |  3.67    5.39
                   0.750   11.90 | 1.750   20.18 | 2.750    7.78 |  3.75    5.09
                   0.833   11.90 | 1.833   20.18 | 2.833    7.78 |  3.83    5.09
                   0.917   17.89 | 1.917   15.63 | 2.917    7.12 |  3.92    4.83
                   1.000   17.89 | 2.000   15.63 | 3.000    7.12 |  4.00    4.83

     Max.Eff.Inten.(mm/hr)=     174.10       108.56
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       1.64 (ii)    5.04 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.32         0.16
                                                        *TOTALS*
     PEAK FLOW       (cms)=       0.27         0.04       0.302 (iii)
     TIME TO PEAK    (hrs)=       1.33         1.42        1.33
     RUNOFF VOLUME    (mm)=      79.66        50.54       73.25
     TOTAL RAINFALL   (mm)=      80.66        80.66       80.66
     RUNOFF COEFFICIENT   =       0.99         0.63        0.91

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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


--------------------
| ADD HYD  (  0002)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0203):     0.50   0.222     1.33    73.83
      + ID2= 2 (  0204):     0.71   0.302     1.33    73.25
        ==================================================
        ID = 3 (  0002):     1.21   0.523     1.33    73.49

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.523) + BASE (  0.550) =   1.073  (cms).
```

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------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0590     0.0998
                          0.0010     0.0484   |   0.0830     0.1754
                          0.0240     0.0726   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0002)      1.210      0.523      1.33      73.49
   OUTFLOW: ID= 1 (  0004)      1.210      0.023      3.00      60.73

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)=  4.33
                   TIME SHIFT OF PEAK FLOW         (min)=100.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.0712

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--------------------
| ADD HYD  (  0005)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0003):     1.19   0.023     2.92    63.19
      + ID2= 2 (  0004):     1.21   0.023     3.00    60.73
        ===============================================
        ID = 3 (  0005):     2.40   0.046     3.00    61.95

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.046) + BASE (  1.100) =   1.146  (cms).
------------------------------------------------------------------------------
********************************
** SIMULATION:4 - Hazel       **
********************************

--------------------
|    READ STORM    |    Filename: C:\Users\CAGW071550\AppD
|                  |              ata\Local\Temp\
|                  |              f57adbc3-d8f6-48d8-bdfe-bd60651496dc\70476bbb
| Ptotal=212.00 mm |    Comments: Hazel
--------------------
                   TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                    hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
                   0.00    6.00 |  3.00   13.00 |   6.00   23.00 |   9.00   53.00
                   1.00    4.00 |  4.00   17.00 |   7.00   13.00 |  10.00   38.00
                   2.00    6.00 |  5.00   13.00 |   8.00   13.00 |  11.00   13.00

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--------------------
| CALIB            |
| STANDHYD (  0201)|   Area    (ha)=   0.69
|ID= 1 DT= 5.0 min |   Total Imp(%)=  88.00   Dir. Conn.(%)=  88.00
--------------------
                            IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.61         0.08
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      67.82        40.00
     Mannings n          =      0.013        0.250

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


                             ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
               0.083    6.00 | 3.083   13.00 | 6.083   23.00 |  9.08   53.00
               0.167    6.00 | 3.167   13.00 | 6.167   23.00 |  9.17   53.00
               0.250    6.00 | 3.250   13.00 | 6.250   23.00 |  9.25   53.00
               0.333    6.00 | 3.333   13.00 | 6.333   23.00 |  9.33   53.00
               0.417    6.00 | 3.417   13.00 | 6.417   23.00 |  9.42   53.00
               0.500    6.00 | 3.500   13.00 | 6.500   23.00 |  9.50   53.00
               0.583    6.00 | 3.583   13.00 | 6.583   23.00 |  9.58   53.00
               0.667    6.00 | 3.667   13.00 | 6.667   23.00 |  9.67   53.00
               0.750    6.00 | 3.750   13.00 | 6.750   23.00 |  9.75   53.00
               0.833    6.00 | 3.833   13.00 | 6.833   23.00 |  9.83   53.00
               0.917    6.00 | 3.917   13.00 | 6.917   23.00 |  9.92   53.00
               1.000    6.00 | 4.000   13.00 | 7.000   23.00 | 10.00   53.00
               1.083    4.00 | 4.083   17.00 | 7.083   13.00 | 10.08   38.00
               1.167    4.00 | 4.167   17.00 | 7.167   13.00 | 10.17   38.00
               1.250    4.00 | 4.250   17.00 | 7.250   13.00 | 10.25   38.00
               1.333    4.00 | 4.333   17.00 | 7.333   13.00 | 10.33   38.00
               1.417    4.00 | 4.417   17.00 | 7.417   13.00 | 10.42   38.00
               1.500    4.00 | 4.500   17.00 | 7.500   13.00 | 10.50   38.00
               1.583    4.00 | 4.583   17.00 | 7.583   13.00 | 10.58   38.00
               1.667    4.00 | 4.667   17.00 | 7.667   13.00 | 10.67   38.00
               1.750    4.00 | 4.750   17.00 | 7.750   13.00 | 10.75   38.00
               1.833    4.00 | 4.833   17.00 | 7.833   13.00 | 10.83   38.00
               1.917    4.00 | 4.917   17.00 | 7.917   13.00 | 10.92   38.00
               2.000    4.00 | 5.000   17.00 | 8.000   13.00 | 11.00   38.00
               2.083    6.00 | 5.083   13.00 | 8.083   13.00 | 11.08   13.00
               2.167    6.00 | 5.167   13.00 | 8.167   13.00 | 11.17   13.00
               2.250    6.00 | 5.250   13.00 | 8.250   13.00 | 11.25   13.00
               2.333    6.00 | 5.333   13.00 | 8.333   13.00 | 11.33   13.00
               2.417    6.00 | 5.417   13.00 | 8.417   13.00 | 11.42   13.00
               2.500    6.00 | 5.500   13.00 | 8.500   13.00 | 11.50   13.00
               2.583    6.00 | 5.583   13.00 | 8.583   13.00 | 11.58   13.00
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           2.667    6.00 | 5.667   13.00 | 8.667   13.00 | 11.67   13.00
           2.750    6.00 | 5.750   13.00 | 8.750   13.00 | 11.75   13.00
           2.833    6.00 | 5.833   13.00 | 8.833   13.00 | 11.83   13.00
           2.917    6.00 | 5.917   13.00 | 8.917   13.00 | 11.92   13.00
           3.000    6.00 | 6.000   13.00 | 9.000   13.00 | 12.00   13.00

     Max.Eff.Inten.(mm/hr)=      53.00        50.33
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       2.61 (ii)    6.71 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.29         0.14
                                                        *TOTALS*
     PEAK FLOW       (cms)=       0.09         0.01       0.101 (iii)
     TIME TO PEAK    (hrs)=       9.67        10.00       10.00
     RUNOFF VOLUME    (mm)=     211.00       173.55      206.50
     TOTAL RAINFALL   (mm)=     212.00       212.00      212.00
     RUNOFF COEFFICIENT   =       1.00         0.82        0.97

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.10) + QBASE(  0.55) =    0.65  (cms)

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--------------------
| CALIB            |
| STANDHYD (  0202)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  90.00   Dir. Conn.(%)=  90.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.45         0.05
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n           =      0.013        0.250

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


                               ---- TRANSFORMED HYETOGRAPH ----
            TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
             hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
           0.083    6.00 | 3.083   13.00 | 6.083   23.00 |  9.08   53.00
           0.167    6.00 | 3.167   13.00 | 6.167   23.00 |  9.17   53.00
           0.250    6.00 | 3.250   13.00 | 6.250   23.00 |  9.25   53.00
           0.333    6.00 | 3.333   13.00 | 6.333   23.00 |  9.33   53.00
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           0.417    6.00 | 3.417   13.00 | 6.417   23.00 |  9.42   53.00
           0.500    6.00 | 3.500   13.00 | 6.500   23.00 |  9.50   53.00
           0.583    6.00 | 3.583   13.00 | 6.583   23.00 |  9.58   53.00
           0.667    6.00 | 3.667   13.00 | 6.667   23.00 |  9.67   53.00
           0.750    6.00 | 3.750   13.00 | 6.750   23.00 |  9.75   53.00
           0.833    6.00 | 3.833   13.00 | 6.833   23.00 |  9.83   53.00
           0.917    6.00 | 3.917   13.00 | 6.917   23.00 |  9.92   53.00
           1.000    6.00 | 4.000   13.00 | 7.000   23.00 | 10.00   53.00
           1.083    4.00 | 4.083   17.00 | 7.083   13.00 | 10.08   38.00
           1.167    4.00 | 4.167   17.00 | 7.167   13.00 | 10.17   38.00
           1.250    4.00 | 4.250   17.00 | 7.250   13.00 | 10.25   38.00
           1.333    4.00 | 4.333   17.00 | 7.333   13.00 | 10.33   38.00
           1.417    4.00 | 4.417   17.00 | 7.417   13.00 | 10.42   38.00
           1.500    4.00 | 4.500   17.00 | 7.500   13.00 | 10.50   38.00
           1.583    4.00 | 4.583   17.00 | 7.583   13.00 | 10.58   38.00
           1.667    4.00 | 4.667   17.00 | 7.667   13.00 | 10.67   38.00
           1.750    4.00 | 4.750   17.00 | 7.750   13.00 | 10.75   38.00
           1.833    4.00 | 4.833   17.00 | 7.833   13.00 | 10.83   38.00
           1.917    4.00 | 4.917   17.00 | 7.917   13.00 | 10.92   38.00
           2.000    4.00 | 5.000   17.00 | 8.000   13.00 | 11.00   38.00
           2.083    6.00 | 5.083   13.00 | 8.083   13.00 | 11.08   13.00
           2.167    6.00 | 5.167   13.00 | 8.167   13.00 | 11.17   13.00
           2.250    6.00 | 5.250   13.00 | 8.250   13.00 | 11.25   13.00
           2.333    6.00 | 5.333   13.00 | 8.333   13.00 | 11.33   13.00
           2.417    6.00 | 5.417   13.00 | 8.417   13.00 | 11.42   13.00
           2.500    6.00 | 5.500   13.00 | 8.500   13.00 | 11.50   13.00
           2.583    6.00 | 5.583   13.00 | 8.583   13.00 | 11.58   13.00
           2.667    6.00 | 5.667   13.00 | 8.667   13.00 | 11.67   13.00
           2.750    6.00 | 5.750   13.00 | 8.750   13.00 | 11.75   13.00
           2.833    6.00 | 5.833   13.00 | 8.833   13.00 | 11.83   13.00
           2.917    6.00 | 5.917   13.00 | 8.917   13.00 | 11.92   13.00
           3.000    6.00 | 6.000   13.00 | 9.000   13.00 | 12.00   13.00

     Max.Eff.Inten.(mm/hr)=      53.00        50.33
                over (min)        5.00        10.00
     Storage Coeff.  (min)=       2.37 (ii)    6.15 (ii)
     Unit Hyd. Tpeak (min)=       5.00        10.00
     Unit Hyd. peak  (cms)=       0.30         0.15
                                                        *TOTALS*
     PEAK FLOW       (cms)=       0.07         0.01       0.073 (iii)
     TIME TO PEAK    (hrs)=       9.58        10.00       10.00
     RUNOFF VOLUME    (mm)=     211.00       173.55      207.25
     TOTAL RAINFALL   (mm)=     212.00       212.00      212.00
     RUNOFF COEFFICIENT   =       1.00         0.82        0.98

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
```

```
          THAN THE STORAGE COEFFICIENT.
    (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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--------------------
| ADD HYD  (  0001)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0201):     0.69   0.101    10.00   206.50
      + ID2= 2 (  0202):     0.50   0.073    10.00   207.25
        ==================================================
        ID = 3 (  0001):     1.19   0.174    10.00   206.82

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.174) + BASE (  0.550) =   0.724  (cms).
--------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0003)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0610     0.1006
                          0.0010     0.0488   |   0.0850     0.1769
                          0.0240     0.0732   |   0.0000     0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0001)      1.190      0.174     10.00     206.82
   OUTFLOW: ID= 1 (  0003)      1.190      0.075     11.08     192.42

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 42.86
                   TIME SHIFT OF PEAK FLOW        (min)= 65.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.1442

--------------------------------------------------------------------------------
--------------------
| CALIB            |
| STANDHYD (  0203)|   Area    (ha)=   0.50
|ID= 1 DT= 5.0 min |   Total Imp(%)=  80.00   Dir. Conn.(%)=  80.00
--------------------
                              IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.40         0.10
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      57.74        40.00
     Mannings n          =      0.013        0.250

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

```
                         ---- TRANSFORMED HYETOGRAPH ----
             TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
              hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
            0.083    6.00 | 3.083   13.00 | 6.083   23.00 |  9.08   53.00
            0.167    6.00 | 3.167   13.00 | 6.167   23.00 |  9.17   53.00
            0.250    6.00 | 3.250   13.00 | 6.250   23.00 |  9.25   53.00
            0.333    6.00 | 3.333   13.00 | 6.333   23.00 |  9.33   53.00
            0.417    6.00 | 3.417   13.00 | 6.417   23.00 |  9.42   53.00
            0.500    6.00 | 3.500   13.00 | 6.500   23.00 |  9.50   53.00
            0.583    6.00 | 3.583   13.00 | 6.583   23.00 |  9.58   53.00
            0.667    6.00 | 3.667   13.00 | 6.667   23.00 |  9.67   53.00
            0.750    6.00 | 3.750   13.00 | 6.750   23.00 |  9.75   53.00
            0.833    6.00 | 3.833   13.00 | 6.833   23.00 |  9.83   53.00
            0.917    6.00 | 3.917   13.00 | 6.917   23.00 |  9.92   53.00
            1.000    6.00 | 4.000   13.00 | 7.000   23.00 | 10.00   53.00
            1.083    4.00 | 4.083   17.00 | 7.083   13.00 | 10.08   38.00
            1.167    4.00 | 4.167   17.00 | 7.167   13.00 | 10.17   38.00
            1.250    4.00 | 4.250   17.00 | 7.250   13.00 | 10.25   38.00
            1.333    4.00 | 4.333   17.00 | 7.333   13.00 | 10.33   38.00
            1.417    4.00 | 4.417   17.00 | 7.417   13.00 | 10.42   38.00
            1.500    4.00 | 4.500   17.00 | 7.500   13.00 | 10.50   38.00
            1.583    4.00 | 4.583   17.00 | 7.583   13.00 | 10.58   38.00
            1.667    4.00 | 4.667   17.00 | 7.667   13.00 | 10.67   38.00
            1.750    4.00 | 4.750   17.00 | 7.750   13.00 | 10.75   38.00
            1.833    4.00 | 4.833   17.00 | 7.833   13.00 | 10.83   38.00
            1.917    4.00 | 4.917   17.00 | 7.917   13.00 | 10.92   38.00
            2.000    4.00 | 5.000   17.00 | 8.000   13.00 | 11.00   38.00
            2.083    6.00 | 5.083   13.00 | 8.083   13.00 | 11.08   13.00
            2.167    6.00 | 5.167   13.00 | 8.167   13.00 | 11.17   13.00
            2.250    6.00 | 5.250   13.00 | 8.250   13.00 | 11.25   13.00
            2.333    6.00 | 5.333   13.00 | 8.333   13.00 | 11.33   13.00
            2.417    6.00 | 5.417   13.00 | 8.417   13.00 | 11.42   13.00
            2.500    6.00 | 5.500   13.00 | 8.500   13.00 | 11.50   13.00
            2.583    6.00 | 5.583   13.00 | 8.583   13.00 | 11.58   13.00
            2.667    6.00 | 5.667   13.00 | 8.667   13.00 | 11.67   13.00
            2.750    6.00 | 5.750   13.00 | 8.750   13.00 | 11.75   13.00
            2.833    6.00 | 5.833   13.00 | 8.833   13.00 | 11.83   13.00
            2.917    6.00 | 5.917   13.00 | 8.917   13.00 | 11.92   13.00
            3.000    6.00 | 6.000   13.00 | 9.000   13.00 | 12.00   13.00

     Max.Eff.Inten.(mm/hr)=      53.00        50.33
                over (min)        5.00        15.00
     Storage Coeff.  (min)=       2.37 (ii)   11.66 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=       0.30         0.09
                                                          *TOTALS*
     PEAK FLOW       (cms)=       0.06         0.01         0.073 (iii)
     TIME TO PEAK    (hrs)=       9.58        10.00         10.00
     RUNOFF VOLUME    (mm)=     211.00       173.55        203.49
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     TOTAL RAINFALL   (mm)=     212.00       212.00         212.00
     RUNOFF COEFFICIENT   =       1.00         0.82           0.96

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.
            ACTUAL QPEAK: PEAK(    0.07) + QBASE(  0.55) =    0.62  (cms)

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--------------------
| CALIB              |
| STANDHYD (  0204)|   Area    (ha)=   0.71
|ID= 1 DT= 5.0 min |   Total Imp(%)=  78.00   Dir. Conn.(%)=  78.00
--------------------
                             IMPERVIOUS    PERVIOUS (i)
     Surface Area     (ha)=       0.55         0.16
     Dep. Storage     (mm)=       1.00         1.50
     Average Slope     (%)=       1.00         2.00
     Length            (m)=      68.80        40.00
     Mannings n          =      0.013        0.250

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


                              ---- TRANSFORMED HYETOGRAPH ----
                TIME    RAIN |  TIME    RAIN |'  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |'   hrs   mm/hr |   hrs   mm/hr
               0.083    6.00 | 3.083   13.00 | 6.083   23.00 |  9.08   53.00
               0.167    6.00 | 3.167   13.00 | 6.167   23.00 |  9.17   53.00
               0.250    6.00 | 3.250   13.00 | 6.250   23.00 |  9.25   53.00
               0.333    6.00 | 3.333   13.00 | 6.333   23.00 |  9.33   53.00
               0.417    6.00 | 3.417   13.00 | 6.417   23.00 |  9.42   53.00
               0.500    6.00 | 3.500   13.00 | 6.500   23.00 |  9.50   53.00
               0.583    6.00 | 3.583   13.00 | 6.583   23.00 |  9.58   53.00
               0.667    6.00 | 3.667   13.00 | 6.667   23.00 |  9.67   53.00
               0.750    6.00 | 3.750   13.00 | 6.750   23.00 |  9.75   53.00
               0.833    6.00 | 3.833   13.00 | 6.833   23.00 |  9.83   53.00
               0.917    6.00 | 3.917   13.00 | 6.917   23.00 |  9.92   53.00
               1.000    6.00 | 4.000   13.00 | 7.000   23.00 | 10.00   53.00
               1.083    4.00 | 4.083   17.00 | 7.083   13.00 | 10.08   38.00
               1.167    4.00 | 4.167   17.00 | 7.167   13.00 | 10.17   38.00
               1.250    4.00 | 4.250   17.00 | 7.250   13.00 | 10.25   38.00
               1.333    4.00 | 4.333   17.00 | 7.333   13.00 | 10.33   38.00
               1.417    4.00 | 4.417   17.00 | 7.417   13.00 | 10.42   38.00
               1.500    4.00 | 4.500   17.00 | 7.500   13.00 | 10.50   38.00
               1.583    4.00 | 4.583   17.00 | 7.583   13.00 | 10.58   38.00
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               1.667    4.00 | 4.667   17.00 | 7.667   13.00 | 10.67   38.00
               1.750    4.00 | 4.750   17.00 | 7.750   13.00 | 10.75   38.00
               1.833    4.00 | 4.833   17.00 | 7.833   13.00 | 10.83   38.00
               1.917    4.00 | 4.917   17.00 | 7.917   13.00 | 10.92   38.00
               2.000    4.00 | 5.000   17.00 | 8.000   13.00 | 11.00   38.00
               2.083    6.00 | 5.083   13.00 | 8.083   13.00 | 11.08   13.00
               2.167    6.00 | 5.167   13.00 | 8.167   13.00 | 11.17   13.00
               2.250    6.00 | 5.250   13.00 | 8.250   13.00 | 11.25   13.00
               2.333    6.00 | 5.333   13.00 | 8.333   13.00 | 11.33   13.00
               2.417    6.00 | 5.417   13.00 | 8.417   13.00 | 11.42   13.00
               2.500    6.00 | 5.500   13.00 | 8.500   13.00 | 11.50   13.00
               2.583    6.00 | 5.583   13.00 | 8.583   13.00 | 11.58   13.00
               2.667    6.00 | 5.667   13.00 | 8.667   13.00 | 11.67   13.00
               2.750    6.00 | 5.750   13.00 | 8.750   13.00 | 11.75   13.00
               2.833    6.00 | 5.833   13.00 | 8.833   13.00 | 11.83   13.00
               2.917    6.00 | 5.917   13.00 | 8.917   13.00 | 11.92   13.00
               3.000    6.00 | 6.000   13.00 | 9.000   13.00 | 12.00   13.00

     Max.Eff.Inten.(mm/hr)=      53.00        50.33
                over (min)        5.00        15.00
     Storage Coeff.  (min)=       2.63 (ii)   11.92 (ii)
     Unit Hyd. Tpeak (min)=       5.00        15.00
     Unit Hyd. peak  (cms)=       0.29         0.09
                                                          *TOTALS*
     PEAK FLOW      (cms)=       0.08         0.02          0.103 (iii)
     TIME TO PEAK    (hrs)=       9.67        10.00          10.00
     RUNOFF VOLUME    (mm)=     211.00       173.55         202.75
     TOTAL RAINFALL   (mm)=     212.00       212.00         212.00
     RUNOFF COEFFICIENT   =       1.00         0.82           0.96

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

       (i) CN PROCEDURE SELECTED FOR PERVIOUS LOSSES:
            CN*  =  85.0    Ia = Dep. Storage  (Above)
      (ii) TIME STEP (DT) SHOULD BE SMALLER OR EQUAL
           THAN THE STORAGE COEFFICIENT.
     (iii) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

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

--------------------
| ADD HYD  (  0002)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0203):     0.50   0.073    10.00   203.49
      + ID2= 2 (  0204):     0.71   0.103    10.00   202.75
        ==================================================
        ID = 3 (  0002):     1.21   0.176    10.00   203.06

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
```

```
     ACTUAL PEAK FLOW: PEAK (  0.176) + BASE (  0.550) =   0.726  (cms).
--------------------------------------------------------------------------------
--------------------
| RESERVOIR(  0004)|     OVERFLOW IS OFF
| IN= 2---> OUT= 1 |
| DT=  5.0 min     |     OUTFLOW    STORAGE   |  OUTFLOW    STORAGE
--------------------      (cms)     (ha.m.)   |   (cms)     (ha.m.)
                          0.0000     0.0000   |   0.0590      0.0998
                          0.0010     0.0484   |   0.0830      0.1754
                          0.0240     0.0726   |   0.0000      0.0000

                                AREA     QPEAK     TPEAK       R.V.
                                (ha)     (cms)     (hrs)       (mm)
   INFLOW : ID= 2 (  0002)      1.210      0.176     10.00     203.06
   OUTFLOW: ID= 1 (  0004)      1.210      0.073     11.08     189.12

                   PEAK   FLOW   REDUCTION [Qout/Qin](%)= 41.74
                   TIME SHIFT OF PEAK FLOW         (min)= 65.00
                   MAXIMUM  STORAGE   USED       (ha.m.)=  0.1451

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--------------------
| ADD HYD  (  0005)|
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (ha)    (cms)    (hrs)     (mm)
        ID1= 1 (  0003):     1.19   0.075    11.08   192.42
      + ID2= 2 (  0004):     1.21   0.073    11.08   189.12
        ====================================================
        ID = 3 (  0005):     2.40   0.148    11.08   190.75

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.
     ACTUAL PEAK FLOW: PEAK (  0.148) + BASE (  1.100) =   1.248  (cms).
--------------------------------------------------------------------------------
```