

**COUNTERPOINT**  
LAND DEVELOPMENT BY

**DILLON**  
CONSULTING

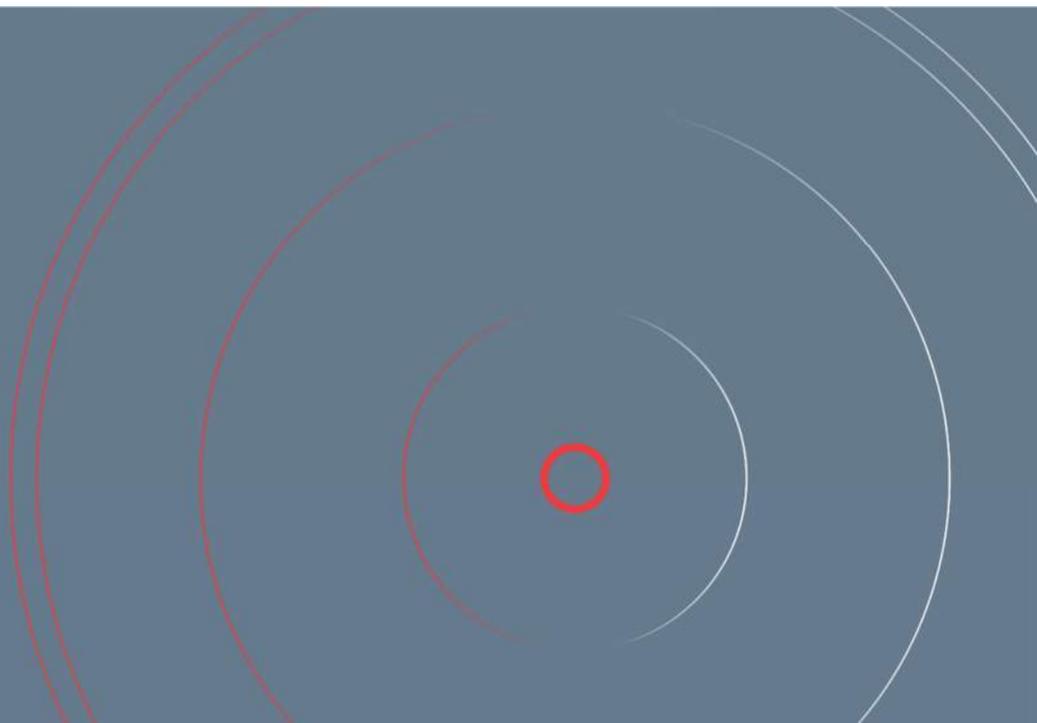
York Regional Police

# FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

350 Garfield Wright Boulevard

Version: 1<sup>st</sup> Submission

August 30, 2024



# EXECUTIVE SUMMARY

This Functional Servicing and Stormwater Management Report ('FSSR') has been prepared to support a Site Plan Approval ('SPA') for the site municipally known as 350 Garfield Wright Boulevard in the Town of East Gwillimbury (referred to as 'the subject site' in this report). The report has been prepared on behalf of York Regional Police (the applicant).

The subject site is currently vacant, and it is proposed to be developed into a helicopter hangar building and parking, an associated vehicle parking, and landscaped areas.

The servicing strategy for the proposed development is summarized as follows:

## **Water Servicing:**

The adjacent municipal roadways contain typical sized watermains to service the proposed development. The domestic and fire flow water demands were calculated in accordance with Town of East Gwillimbury criteria and the Fire Underwriter's Survey methodology. The maximum day demand plus fire flow demand is 12,026 L/min.

A hydrant flow test will need to be conducted on the existing watermain on Garfield Wright Boulevard to confirm available watermain pressure at the required flow rates.

## **Sanitary Servicing:**

The subject site is located outside the urban service boundary of the Town of East Gwillimbury and there are no sanitary sewers in the vicinity of the subject site. The proposed hangar building will be serviced by a private on-site septic system consisting of a Waterloo anaerobic digester, a Waterloo Biofilter, and an in-ground dispersal bed. The sanitary daily design flow for the subject site is 3,629 L/day and has been calculated in accordance with Part 8 of the Ontario Building Code.

## **Stormwater Servicing & Stormwater Management:**

The subject site is located within the York Region Industrial Subdivision (YRIS) SWM facility service area. The YRIS SWM facility provides quality, erosion control, and quantity control for the entire catchment area of YRIS. Stormwater quantity control for the development area will be provided by the downstream YRIS SWM facility. The downstream YRIS SWM facility will provide water balance, erosion control, and stormwater quality control (Enhanced Level 1 / 80% TSS removal). Per the Lake Simcoe Region Conservation Authority Phosphorus Budget Guidance Tool, the pre-development phosphorus loading is 0.05 kg/year. The unmitigated post-development phosphorus loading is 1.22 kg/year. To mitigate the phosphorus loading to levels below the pre-development conditions, a treatment train approach consisting of an upstream infiltration trench and a Jellyfish filter unit to reduce the phosphorus loading to 1.22 kg/year.

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- Percolation Test Report & Test Pit Plan

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- MECP PBGT output
- Jellyfish Filter ETV verification statement
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- Engineering Drawings prepared by Counterpoint Engineering Inc.
- Site Plan Drawing, prepared by Parkin Architects Limited

# 1.0 INTRODUCTION

## 1.1 BACKGROUND

This Functional Servicing and Stormwater Management Report (FSSR) has been prepared to support the development of the new York Regional Police (YRP) hangar to be located within York Regional lands located east of Bales Drive East in the Town of East Gwillimbury. Dillon Consulting Limited (Counterpoint Land Development) has been retained by Parkin Architects Limited to prepare the SWM and functional site servicing/grading plans of the proposed development.

The subject site is located on the north side of Garfield Wright Boulevard and is bounded by the existing York Region Industrial Subdivision (YRIS) SWM facility to the east, the existing gravel parking lot to the north and the existing septic bed/open space to the west. Refer to **Figure 1** for the Site Location Plan.

The proposed development will consist of a one-story building consisting of office space and helicopter parking, a helicopter landing pad and parking areas, driveways and a parking lot, and associated landscaped areas.

## 1.2 STUDY PARAMETERS

This functional servicing assessment for the subject site is based on the review of the following documents and drawings:

- Architectural site plan prepared by Parkin Architects Limited
- Technical Design Brief – Stormwater Management Facilities for York Regional Industrial Subdivision 19T-94016; July 2004 prepared by Cumming Cockburn Limited (CCL).
- Storm Drainage Area Plan (Drawing 5390-STM2) for the YRIS storm sewers prepared by CCL and dated May 2004.
- Ontario Building Code, Section 8
- Town of East Gwillimbury Engineering Standards and Design Criteria, September 2012
- Lake Simcoe Region Conservation Authority Technical Guidelines for Stormwater Management Submissions April 2022
- Lake Simcoe Region Conservation Authority Black River Subwatershed Plan
- Ministry of the Environment, Conservation and Parks Design Guidelines for Drinking Water Systems, January 2016
- Stormwater Management Implementation Report for 2696 & 2740 Davis Drive Industrial Development, dated July 2013 prepared by RJ Burnside & Associates Ltd.
- Servicing and Grading Plans for the Technicore Industrial Subdivision, dated February 2007 and prepared by RJ Burnside.
- Determination of Estimated T-Time, dated July 31, 2016, and prepared by Azimuth Environmental Consulting Inc./GEI Consultants



<b>counterpoint</b> <small>ENGINEERING</small> <small>A SUBSIDIARY OF BILLOM CONSULTING LIMITED</small> <small>8395 Jane St., Suite 100, Vaughan, ON L4K 5Y2 Phone 905.326.1404 Fax 905.326.1405</small>		<b>SITE LOCATION PLAN</b>	
		DESIGNED BY: PM	DATE: FEB 08, 2023
		CHECKED BY: PM	PROJECT No. <b>24015</b>
		DRAWING BY: PM	
<b>YRP HELICOPTER HANGAR</b>		CHECKED BY: PM	FIGURE No. <b>1</b>
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## 2.0 WATER SUPPLY

### 2.1 EXISTING WATER SUPPLY

There is an existing 300 mm $\varnothing$  watermain on the south side of Garfield Wright Boulevard. Refer to **Figure 2 - Water Servicing Plan** for the existing watermain layout. There are two municipal hydrants located on the south side of the road allowance. There are no existing water service services within the subject site.

Municipal water service is currently provided to the Town of East Gwillimbury and some other areas, distributed through a network of water mains with diameters up to 400 mm. The remainder of the municipality relies on individual wells for their water needs. The municipal water system depends on a groundwater supply from local wells.

### 2.2 WATER DEMAND

#### 2.2.1 DOMESTIC DEMAND

Calculation of the water demand for the proposed development has been performed using the guidelines outlined within the Town of East Gwillimbury Engineering Standards and Design Criteria (September 2012), and the MECP Design Guidelines for Drinking Water Systems (2016).

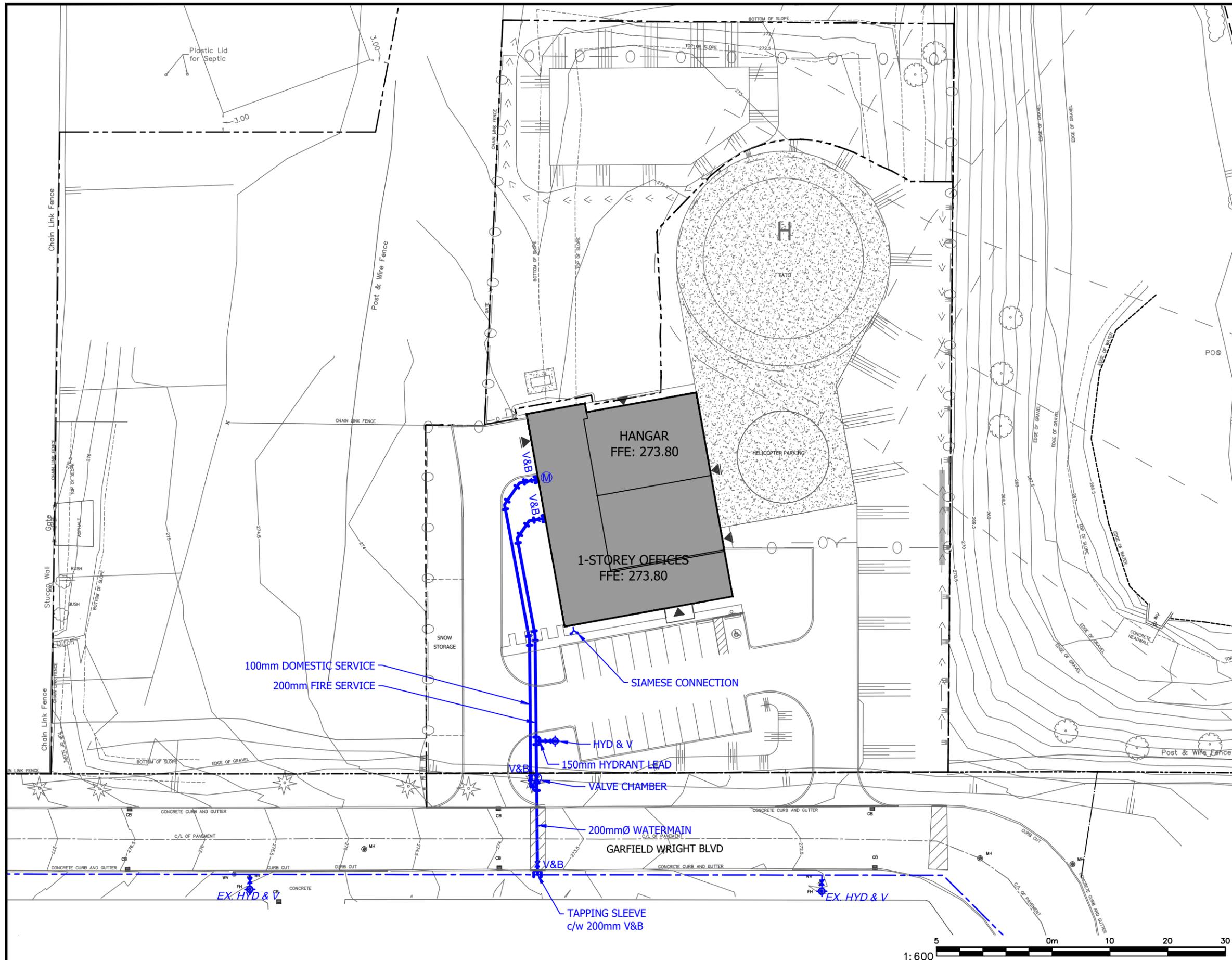
Per the Town of East Gwillimbury, the average day demand for commercial land use is 28,000 L/ha/day. Maximum Day and Peak Hour factors shall be 2.0 and 2.75 respectively, or as recommended by the MECP.

Refer to **Appendix A** for the supporting calculations of the following proposed domestic demands:

- Maximum Hour Demand = 35.8 L/min
- Maximum Day Demand = 26.0 L/min

#### 2.2.2 FIREFLOWS

The fire flow required for the proposed hangar building has been calculated using the criteria indicated in the Water Supply for Public Fire Protection Manual, 1999, by the Fire Underwriters Survey (FUS). The calculation incorporates various parameters such as coefficient for fire-resistant construction, area reduction accounting for a fire-resistant (one-hour rating) protection, reduction for low-hazard occupancies, adjustment for sprinkler protection system, and factor for neighbouring building proximity. Based on the calculations, the minimum fire suppression flow required is 4,725 L/min. This fire flow plus the maximum day demand or peak hour demand, whichever is greater, must be available at the nearest hydrant with a minimum pressure of 140 KPa. Refer to **Appendix A** for the supporting calculations of the following proposed fireflows:



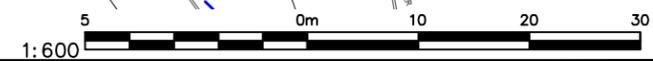
**LEGEND**

- PROPERTY LINE
- PROPOSED BUILDING EVELOPE
- EXISTING WATER SUPPLY
- PROPOSED WATER SUPPLY
- B PROPOSED BACKFLOW PREVENTER
- M PROPOSED WATER METER
- PROPOSED VALVE AND BOX
- SIAMESE CONNECTION

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350 GARFIELD WRIGHT BOULEVARD  
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<b>WATER SERVICING PLAN</b>	
DESIGNED BY: PM	DATE: APRIL 2024
CHECKED BY: PW	PROJECT No. 24015
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- Fire Flow Demand (2 hours) = 12,000 L/min
- Maximum Day Demand plus Fire Flow Demand = **12,026 L/min**

In accordance with the FUS fire flows for the existing watermain on Garfield Wright Boulevard will not be less than **12,026 L/min** for a 2-hour duration in addition to the maximum daily domestic demand, delivered with a residual pressure of not less than 140 kPa.

A hydrant flow test will need to be conducted on the existing watermain on Garfield Wright Boulevard to confirm available watermain pressure at the required flow rates.

## 2.3 WATER SERVICE CONNECTION

It is proposed to install a 200mm diameter water service for the building connecting to the existing 300mm diameter municipal watermain on Garfield Wright Boulevard. This water connection will branch to a separate 100mm diameter domestic watermain and 200mm fireline. A valve chamber will be provided on the fireline at the streetline. The domestic water service and fire line will enter the hangar building from the west side at the mechanical room as indicated on the **Figure 2 – Water Servicing Plan**.

Fire protection will be provided by a proposed private site fire hydrant, which is located on the south side of the proposed hangar building. The fire hydrant will be located within 90m of the principal building entrances and 45m of the proposed Siamese connection in accordance with the Ontario Building Code. The location of the fire hydrants and Siamese connection are indicated on the **Figure 2 - Water Servicing Plan**.

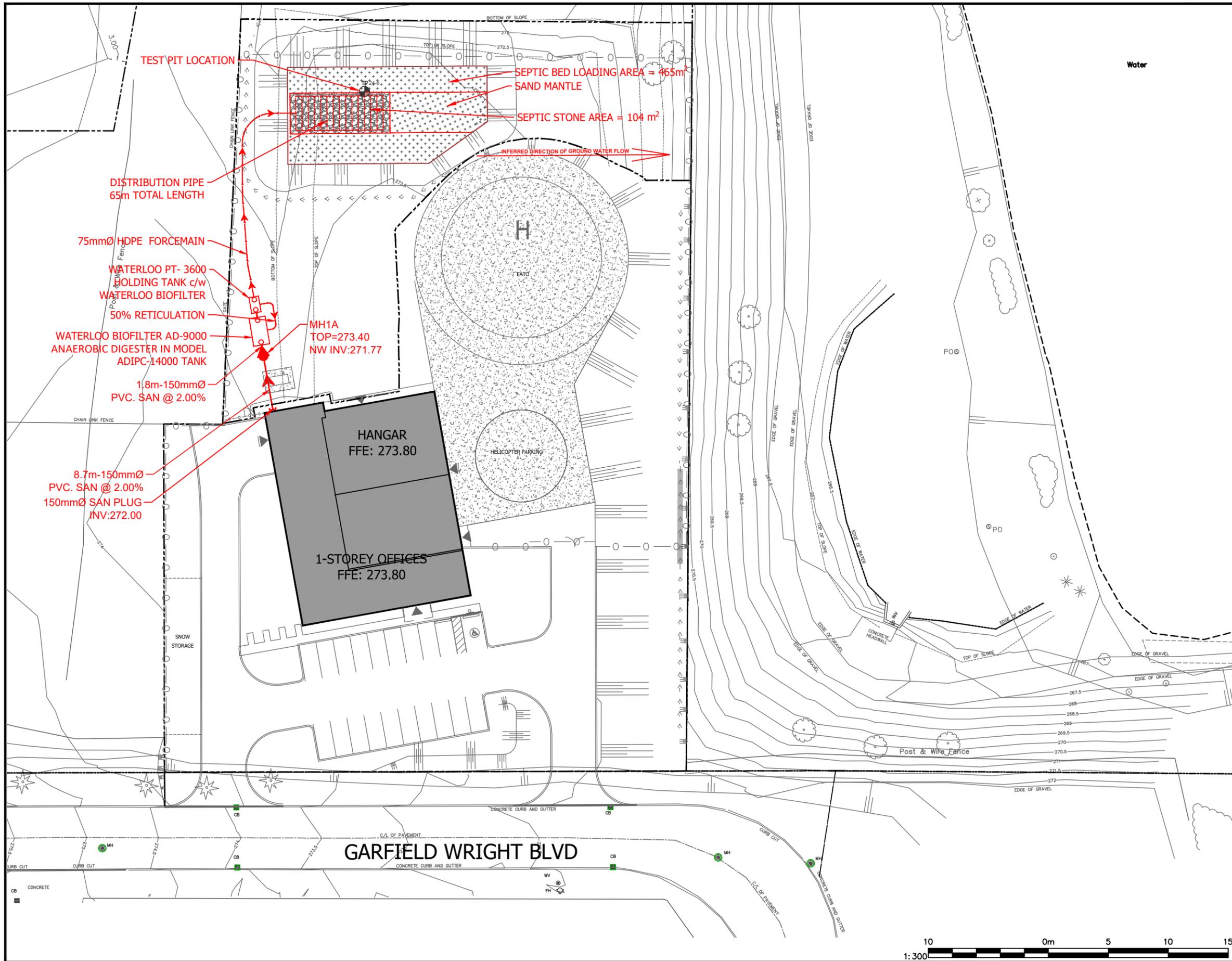
## 3.0 WASTEWATER SERVICING

Given that the subject site is located outside the urban service boundary of the Town of East Gwillimbury, and that there are no sanitary sewers in the vicinity of the subject site, the proposed hangar building will be serviced by a private on-site septic system similar to the adjacent developments within the YRIS. In this regard, the proposed hangar building will be serviced by a septic system which consists of a Waterloo anaerobic digester, a holding tank housing a Waterloo biofilter, and an in-ground dispersal bed as indicated on the **Figure 3 – Sanitary Servicing Plan**.

### 3.1 SANITARY DEMAND

The daily design flow for a septic system is to be calculated in accordance with Part 8 of the OBC. For non-residential uses, the flow is calculated based on the most applicable occupancy as listed in OBC Table 8.2.1.3.B – office @ 450m<sup>2</sup>.

Based on the office occupancy and floor area, the peak daily design flow is 3,629 L/day. Refer to the Sanitary Design Calculation included in **Appendix B**.



## 3.2 SEPTIC SYSTEM COMPONENTS

The septic system is comprised of several components which are to be sized based on the daily design flow and the percolation rate ("T" time) of the native soil conditions. The configuration of the proposed septic system is illustrated in **Figure 3 – Sanitary Servicing Plan** and the sizing of the components is summarized in the following sections.

### 3.2.1 SEPTIC TANK

A gravity sanitary drain will convey sewage flows to the septic tank from the proposed hangar building via a sanitary manhole. In accordance with the manufacturer's design and installation guide, the treatment unit volume is to be a minimum of the daily design flow as follows:

- Minimum Tank Size = 3,629 L/day
- Selected Treatment Unit Size = 4,000 L (Anaerobic Digester – AD-9000)

In accordance with the manufacturer's recommendations, an effluent filter is to be installed in the outlet of the septic tank. Access risers over the tank's inlet and outlet will extend to the finished grade for inspection and maintenance. Effluent from the septic tank will drain to the pump tank.

### 3.2.2 FLOW BALANCING

Given the relatively low sanitary daily design flow and intended use of the hangar building, sanitary flows will be predictable with minimal variability from day to day. As such flow balancing will not be required.

### 3.2.3 TERTIARY TREATMENT UNIT - WATERLOO BIOFILTER

In order to minimize the land area required for the disposal bed and to achieve a level of treatment higher than that of a conventional septic system, the use of an alternative treatment system has been considered.

There are several alternative treatment systems available that have been approved by the recognized by the Ontario Building Code. Alternative treatment systems designed as "Treatment Units" other than septic tanks must meet the requirements of Section 8.6.2.2 of the OBC, must produce either secondary or tertiary quality effluent, and must have received authorization from the Building Materials Evaluation Commission (BMEC). One such technology providing tertiary treatment is the Waterloo Biofilter<sup>®</sup> manufactured by Waterloo Biofilter Systems Inc.

The Waterloo Biofilter is an aerobic trickling filter that uses an absorbent synthetic filter material. Septic tank effluent is applied intermittently over modules of plastic foam pieces (patented biofilter medium) contained in wire mesh baskets. This synthetic media supports microbiological growth, and these microorganisms are responsible for the aerobic breakdown of the wastewater. Approximately 50% of the effluent exiting the unit is pumped back to the septic tank, while the other half is directed to a disposal bed.

In accordance with the manufacturer’s guidelines, based on a balanced design flow of 3,629 L/day the Waterloo Biofilter Basket Tank System (BT-9000) is required. Details of the Waterloo Biofilter® are contained in **Appendix B** together with standard details for the anaerobic digester.

### 3.2.4 SUBSURFACE DISPOSAL

The BMEC authorization for the Waterloo Biofilter permits the use of an area bed dispersal system which is implemented in the majority of installations. The area bed is to be comprised of a stone layer overlying a sand layer where the stone layer is to be a minimum of 300 mm in depth, wrapped with a permeable geo-textile fabric, and comprised of stone meeting the requirements of the OBC. Distribution pipes having 75 mm diameter are to be spaced evenly within the stone layer with spacing not exceeding 1.2 m. The sand layer is to be a minimum of 600 mm in depth below the stone layer and 300 mm above the stone layer.

A field percolation test was conducted by Azimuth Environmental Consulting Inc. which determined that the native soils in the vicinity of the proposed area bed (TP 24-6) as silt, some sand, some clay with trace gravel. The investigation determined the soils have a percolation rate of a percolation rate (“T” time) of 50 min/cm. The field percolation test is included in **Appendix B**.

#### Stone Layer:

Given that the daily design flow is more than 3,000 L, the loading on the surface of the stone layer is calculated as follows:

$$\text{Minimum Surface Area} = Q / 50 = 3,629 / 50 \text{ L} = 72.58 \text{ m}^2$$

$$\text{Design Surface Area} = 6.5\text{m} \times 16\text{m} = 104 \text{ m}^2$$

#### Sand Contact Area:

Given that the “T” time of the native soil greater than 15 min/cm the sand layer is to extend over an area which is calculated as follows:

$$\text{Minimum Surface Area} = Q \times T / 400 = 3,629 \text{ L} \times 50 \text{ min/cm} / 400 = 453.62 \text{ m}^2$$

$$\text{Design Surface Area} = 465.73 \text{ m}^2 \text{ (Refer to **Figure 3- Sanitary Servicing Plan**)}$$

## 4.0 STORM DRAINAGE

The subject site is located within the York Region Industrial Subdivision (YRIS) SWM facility service area. The details of the design of this SWM facility are provided in the report titled “ Technical Design Brief – Stormwater Management Facilities for York Regional Industrial Subdivision 19T-94016” which was prepared by CCL. See **Appendix C**. Based on that report, the Town of East Gwillimbury and the LSRCA have both reviewed the design concept of the SWM facility and drainage area, provided comments, and approved the design.

The YRIS SWM facility has the following characteristics:

- It provides quality, erosion control, and quantity control for the entire catchment area of YRIS (approximately 29 Ha as indicated on Figure 7 of the CCL Technical Design Brief).
- It is designed for enhanced Level 1 quality control.
- It is designed to provide extended detention of a 25 mm storm for 24 hours for erosion control.
- It is designed to provide post-to-peak flow control for storms ranging from a 2-year design event to a 100-year design event.

The design brief of the YRIS SWM facility notes that the SWM approach of the site plans within the service area will be addressed through the site plan approval process and East Gwillimbury Stormwater Master Plan and the Lake Simcoe Protection Plan in addition to the YRIS SWM facility design requirements.

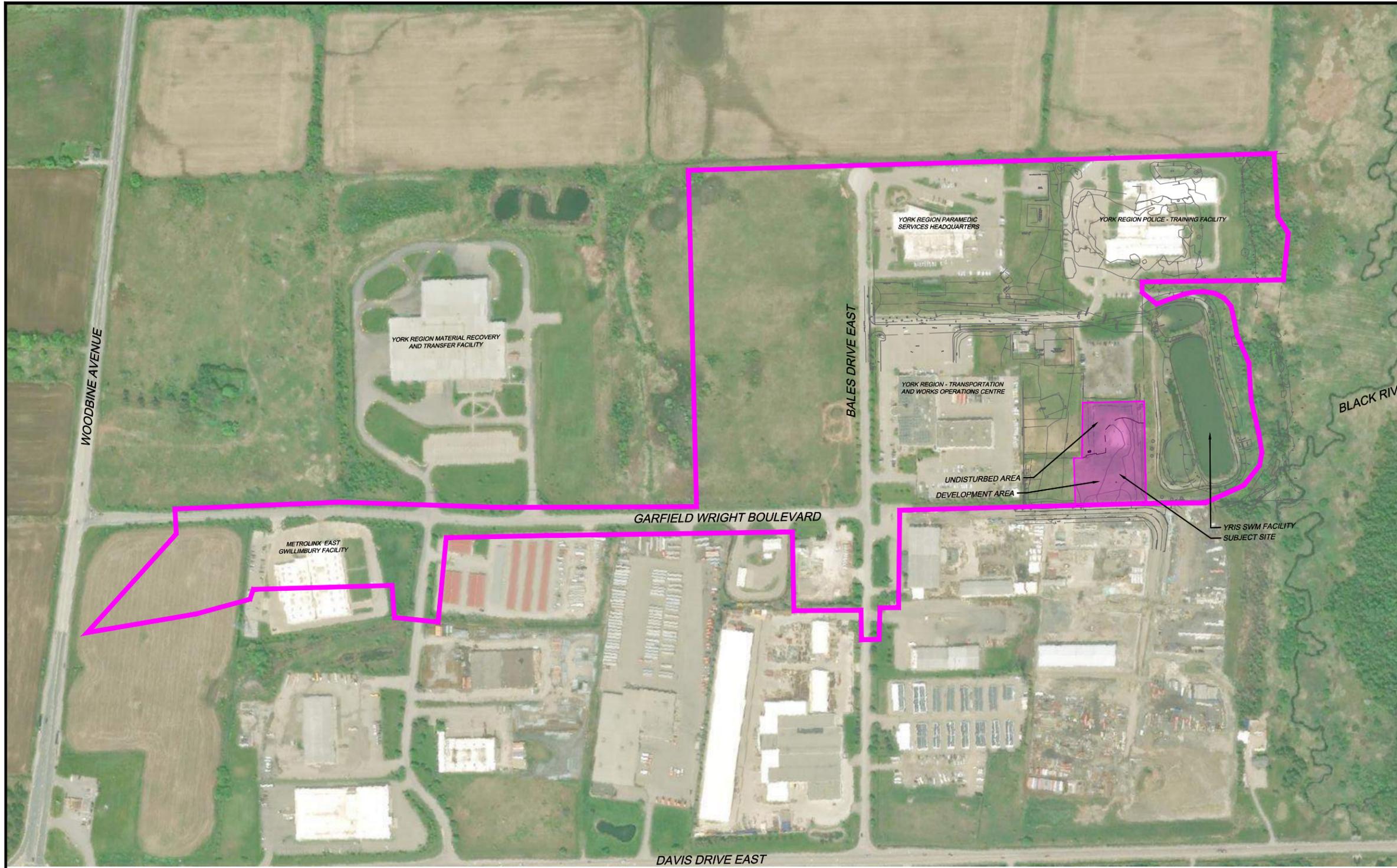
Refer to **Figure 4 – YRIS SWMF Drainage Plan** for the location of the subject site within the extent of the drainage area of the YRIS SWM facility.

## 4.1 EXISTING CONDITIONS & WATERSHED

The subject site is located in the Lake Simcoe watershed, which is under the jurisdiction of the Lake Simcoe Region Conservation Authority (LSRCA). Specifically, the site is situated in the Black River Subwatershed, which occupies 375 km<sup>2</sup> of land south of the eastern portion of Lake Simcoe. The headwaters of the Black River originate on the Oak Ridges Moraine, and the river's watercourses flow mainly through natural features and agricultural areas throughout much of the system before reaching the community of Sutton and ultimately draining into Lake Simcoe. The Subwatershed supports a high level of natural features as well as agricultural activities. Its jurisdiction is primarily within York Region, with a small portion extending into Durham Region. The municipalities within its boundaries include Georgina, East Gwillimbury, Whitchurch-Stouffville, and Uxbridge.

Based on a review of the topographic survey which was provided by Parkin and prepared by Lloyd & Purcell Ltd. in 2014 and updated in 2024, the majority of the subject site can be described as having a gently sloping towards the east while a small portion of the subject site slopes towards the north. Drainage from the major portion of the subject site sheet flows directly to the YRIS SWM facility (south forebay) while drainage from the minor portion is collected by a swale and conveyed to the north forebay. The boulevard appeared to be unfinished and did not drain onto the roadway as per the typical subdivision design standard.

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**LEGEND**

-  PROPERTY LINE
-  YRIS SWM FACILITY DRAINAGE AREA BOUNDARY
-  SUBJECT SITE

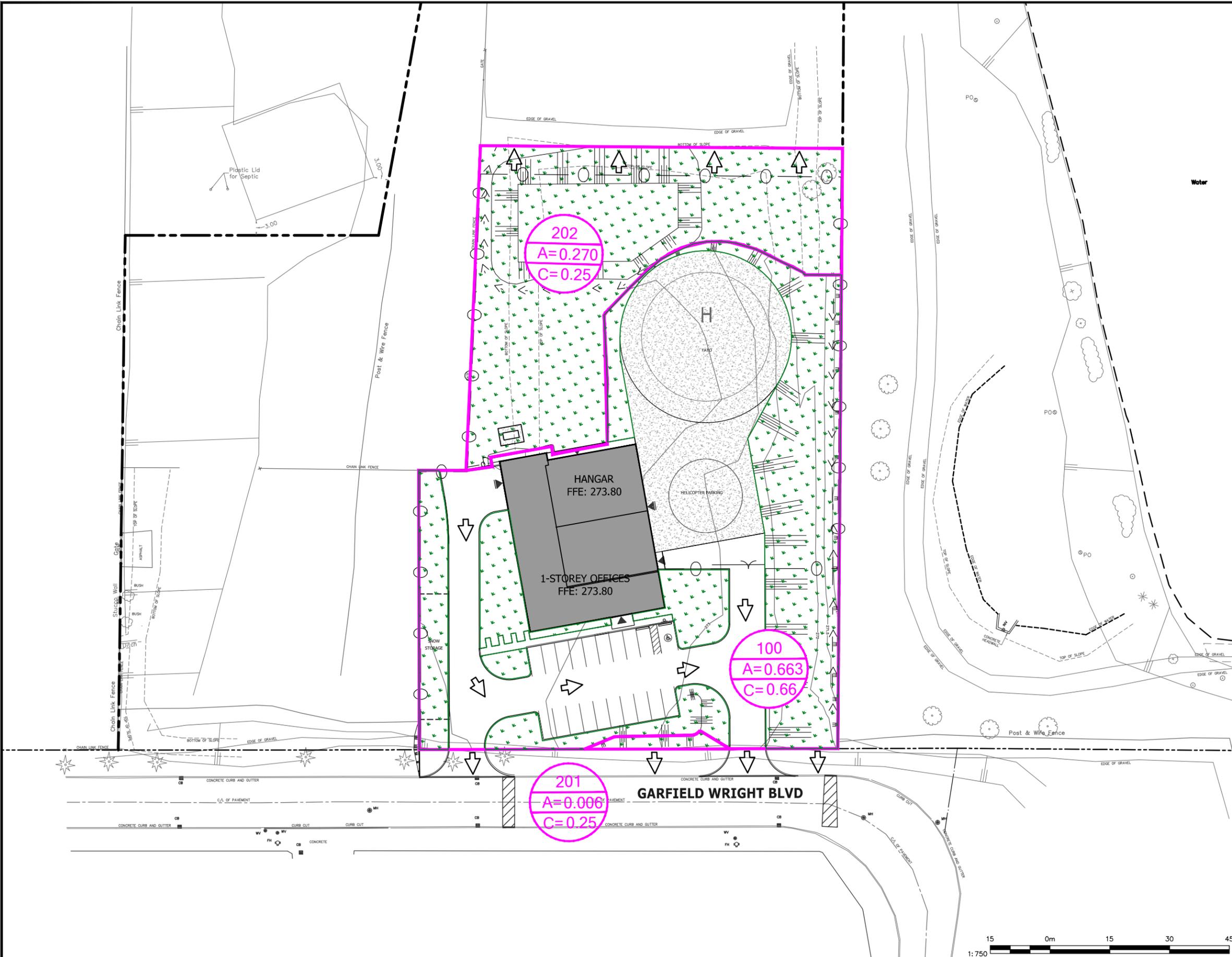
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**YRIS SWMF DRAINAGE PLAN**

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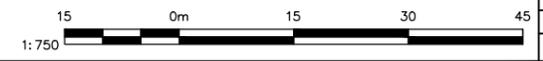
- LEGEND**
- PROPERTY LINE
  - LIMIT OF DEVELOPMENT
  - AREA ID  
AREA (ha)  
RUNOFF COEFFICIENT
  - PERVIOUS AREA
  - OVERLAND FLOW

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**STORMWATER DRAINAGE PLAN**

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## 4.2 STORMWATER MANAGEMENT CRITERIA

The following stormwater management criteria have been established for the subject site based on the review of the YRIS SWM facility design brief. The design criteria will conform to the requirements of the LSRCA Watershed Development Policies and Town of East Gwillimbury Storm Drainage & SWM Design Criteria.

- Quantity Control: to be addressed by the YRIS SWM facility, based on site imperviousness of 80% (R = 0.75)
- Quality Control: to be addressed by the downstream YRIS SWM facility (Enhanced Level 1/ 80% TSS removal)
- Water Balance & Erosion Control: to be addressed by the downstream YRIS SWM facility.
- Phosphorus Loading: target onsite removal of 80% of the annual total phosphorus (TP) load from all major development areas through onsite measures.

## 4.3 STORMWATER RELEASE RATES

In the post development condition, the subject site is divided into two major areas – development area where impervious areas – asphalt, building and walkways are to be introduced and non-development areas where the existing condition (landscaped) will be maintained). Refer to **Figure 5 – Stormwater Drainage Plan** for the delineation of the post development drainage areas.

As noted in **Section 4.2**, quantity control for the entire site will be provided by the downstream YRIS SWM facility. As indicated on **Figure 5 – Stormwater Drainage Plan**, the post development runoff coefficient for the development area is 0.66 which is lower than the 0.75 as per the YRIS SWM facility design. No additional stormwater runoff attenuation is necessary.

The stormwater release rates have been calculated for the development area and are indicated on **Table 1** below and included in **Appendix C**.

**Table 1: Stormwater Release Rates**

POST-DEVELOPMENT DRAINAGE AREA	RECEIVING SYSTEM	5-YEAR RELEASE RATE (L/S)	100-YEAR RELEASE RATE (L/S)
Area 100 – Development Area	YRIS SWM Pond Municipal via storm sewer	139	21.2
Area 101 – Development Area	YRIS SWM Pond via Existing overland flow channel	312	52.8

## 4.4 PROPOSED STORM SERVICING

Stormwater from the 'development area' portion of the subject site will be captured by catch basins and conveyed internally through the site via storm sewers. The internal storm sewer system will be designed to convey the 5-year storm event in accordance with the Town of East Gwillimbury Engineering Standards and Design Criteria. DICB 101 and storm sewer lead will be designed to runoff from storm exceeding the 5-year storm event (up to the and including the 100-year event). Refer to **Figure 6 – Storm Servicing Plan** for the layout of the internal storm sewer system.

## 4.5 STORMWATER QUANTITY CONTROL

As noted in **Section 5.2** stormwater quantity control for the entire site (development + non development areas) will be provided by the downstream YRIS SWM facility. A Jellyfish filter unit will be provided upstream of the control manhole to provide further quality control and phosphorus removal that is discussed in **Section 5.7**. Refer to **Figure 6 – Storm Servicing Plan** for the location of the YRIS SWM facility.

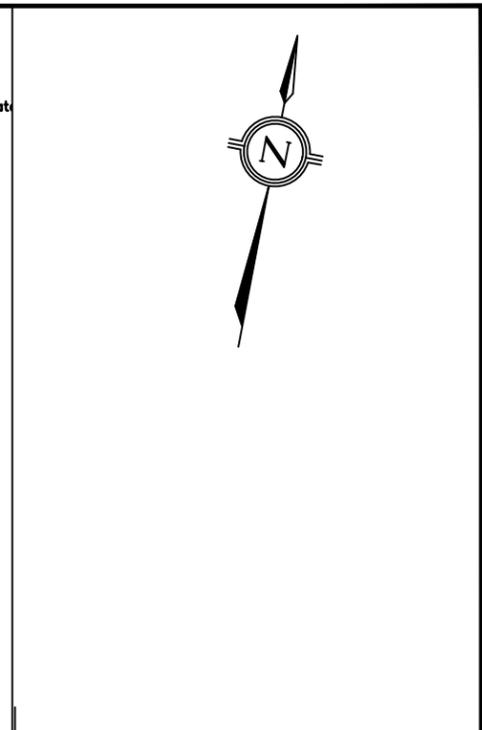
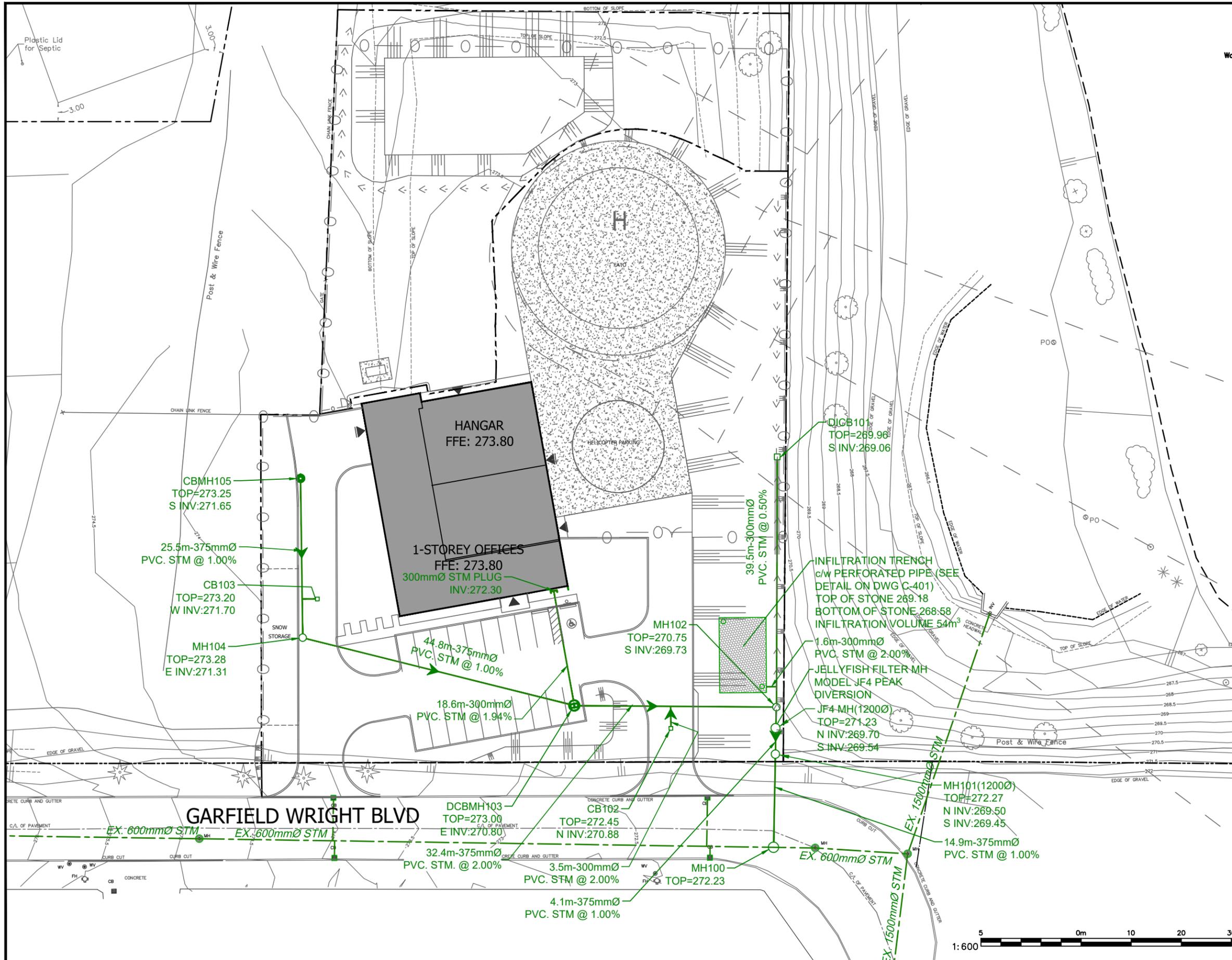
## 4.6 WATER BALANCE & STORMWATER QUALITY CONTROL

The downstream YRIS SWM facility will provide water balance, erosion control and stormwater quality Control (Enhanced Level 1 / 80% TSS removal). Refer to **Figure 6 – Storm Servicing Plan** for the location of the YRIS SWM facility.

## 4.7 PHOSPHORUS CONTROL

Lake Simcoe is enriched by nutrients from land use activities in its watershed and has, for many years, been the focus of efforts to protect and restore its water quality. In 2009, the Lake Simcoe Protection Plan (LSPP) was approved to regulate the input of nutrients, specifically phosphorus, into Lake Simcoe. The intent of the policies in the LSPP is for new development in the Lake Simcoe watershed to adopt BMP, LID techniques, and innovative stormwater management approaches to achieve sustainable development practices that reduce phosphorus loading from new urban development. In this regard, the policy requires that post-development loadings on any major development site be reduced from pre-development loadings.

To establish a method for quantifying and comparing pre- and post-development phosphorus loadings that reflect differing precipitation patterns, soils, and slopes across the Lake Simcoe watershed, the MECP released the Phosphorus Budget Guidance Tool (PBG) to guide new development in the Lake Simcoe watershed in 2012. PBGT uses estimates of phosphorus export developed for specific land uses, coupled with standard estimates of phosphorus reduction efficiencies for various BMPs and LID techniques.



- LEGEND**
- PROPERTY LINE
  - PROPOSED BUILDING EVELOPE
  - EXISTING STORMWATER SEWER
  - PROPOSED STORMWATER SEWER
  - EXISTING STORM MANHOLE
  - PROPOSED STORM MANHOLE
  - EXISTING CATCHBASIN
  - PROPOSED CATCHBASIN
  - PROPOSED AREA DRAIN

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<b>STORM SERVICING PLAN</b>	
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#### 4.7.1 PRE-DEVELOPMENT LOADING

Under pre-development conditions, based on the land use categories of the PBGT the 0.6691 Ha site development area is considered to be partially “hay/pasture”. Based on this category, the PBGT indicates that the pre-development phosphorus loading is 0.05 kg/year as summarized in **Table 2**.

The PBGT output for the pre-development loading is provided in **Appendix C** together with tables listing the various coefficients and land use categories.

#### 4.7.2 POST-DEVELOPMENT LOADING

Under post-development conditions, based on the land use categories of the PBGT, the site is “High Intensity Development – Commercial”. Based on this category, the PBGT indicates that the unmitigated post-development phosphorus loading is 1.22 kg/year.

The results summarized in **Table 2** indicate that there will be an increase decrease of 1.17 kg/year and therefore mitigation measures are required. The PBGT output for the post-development loading is provided in **Appendix C**.

**Table 2: Phosphorus Loading Summary**

LAND USE	AREA (HA)	PHOSPHORUS COEFFICIENT (KG/HA)	PHOSPHORUS LOAD (KG/YR)
<b>PRE-DEVELOPMENT</b>			
Hay/Pasture	0.6691	0.08	0.05
<b>POST DEVELOPMENT</b>			
<b>High Intensity Development – Commercial</b>	0.6691	1.82	1.22
SUBTOTAL (Before TP mitigation)			1.22
Infiltration Trench (c/w perforated pipe) will provide 87% TP removal from controlled areas@ 1.82 kg/Ha.	0.6691	1.82	0.73
TOTAL TP remaining (after infiltration trench mitigation)			0.16
Jellyfish filter will remove 77% of TP remaining after infiltration trench mitigation.	0.6691	1.82	0.12
TOTAL TP remaining (after infiltration trench + Jellyfish filter mitigation)			0.04

To mitigate the phosphorus loading to levels below the pre-development conditions, a treatment train approach consisting of an upstream infiltration trench (c/w perforated pipe) and Jellyfish

filter (housed in a precast manhole) will be implemented. LSRCA credits Perforated Pipe Infiltration/Exfiltration Systems with 87% TP removal while the Jellyfish filter is Environmental Technology Verification (ETV) certified for 77% TP removal. Refer to the ETV certification statement for the Jellyfish filter included in **Appendix C**. The infiltration trench will be situated upstream of the Jellyfish filter unit. Refer to **Figure 6 – Storm Servicing Plan** for the location of the infiltration trench and Jellyfish filter unit.

## 5.0 EROSION AND SEDIMENT CONTROL

Construction activity, especially operations involving topsoil stripping and bulk earthworks dramatically increases the availability of particulate matter for erosion and transport by surface drainage. To mitigate the adverse environmental impacts caused by the release of silt-laden stormwater runoff into receiving watercourses, measures for erosion and sediment control (ESC) are required for construction sites. The Erosion & Sediment Control Guidelines for Urban Construction, December 2006 will guide the selection of the proposed ESC measures. Control measures must be selected that are appropriate for the erosion potential of the site and they must be implemented and modified on a staged basis to reflect the site activities. Furthermore, their effectiveness decreases with sediment loading and therefore regular inspection and maintenance are required. The following ESC measures are proposed:

### 5.1 SILT FENCE

Silt Fences are to be installed adjacent to all grading limits to protect the development area prior to topsoil stripping and in other locations, such as at the bases of topsoil stockpiles. It is recommended that earthworks not extend immediately adjacent to the silt fence and instead 1m to 2 m vegetated buffer be maintained for additional protection. The silt fences are to be constructed with 150 x 150 mm heavy-duty wire farm fence fabric to properly support the geotextile. A heavy-duty silt fence which involves two fences with a straw bale between is recommended to be installed in the vicinity of the buffer area/valleyland.

### 5.2 MUDMAT

A mud mat is to be installed at the construction entrance prior to commencing earthworks to minimize the tracking of mud onto municipal roads. The mud mat will be installed at the location of the existing site entrance on Garfield Wright Boulevard.

### 5.3 SEDIMENT TRAPS

Sediment traps are to be installed at all catchbasin locations once the storm sewer system has been constructed to prevent silt laden runoff from entering. These sediment traps are comprised of clear stone and filter fabric over the catchbasin grate.

## 5.4 STORM SEWER BULKHEADS

Bulk heads are to be installed in the storm sewer manholes at key locations to provide additional sediment control of storm runoff prior to being conveyed to the receiving SWMF. The temporary bulkhead will be installed to the springline elevation of the storm sewer and will remain in place until all surfaces are stabilized. All accumulated sediment is to be removed from the manhole prior to removing the bulkhead.

## 6.0 CONCLUSIONS

This Functional Servicing and Stormwater Management Report presents a site servicing strategy for the proposed development that addresses the requirements of the applicable design guidelines and provides the basis for detailed servicing design.

We trust this report sufficiently addresses the site servicing requirements and allows for approval of the proposed SPA application with respect to the subject site for the proposed use described herein. Should there be any questions or comments, please feel free to contact the undersigned.

Sincerely,

Counterpoint Land Development by Dillon Consulting Limited



Pula Mathumo, P.Eng  
Project Engineer/Manager  
Email: [pmathumo@counterpointeng.com](mailto:pmathumo@counterpointeng.com)

### **Terms of Use**

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# APPENDIX A

## Domestic and Fire Demand Calculations

**Water Demand Design Calculations**

**Project:** 350 Garfield Wright Boulevard  
**Project No:** 24015  
**Client:** York Regional Police  
**Location:** East Gwillmbury, Ontario  
**Site Area:** 0.67 ha (development area only)

**Prepared by:** PM  
**Checked by:** PT  
**Last Revised:** 29-Aug-24

**Domestic Demand per Landuse**

Industrial	35,000	Litres/Ha/day	*Applicable to subject site
Commercial	28,000	Litres/Ha/day	
Institutional	18,000	Litres/Ha/day	

**Per Capita Demand**

Average flow	350 litres/person/day
--------------	-----------------------

**Peaking Factors**

Land Use	Minimum Hour	Maximum Hour	Maximum Day
Commercial	0.40	2.75	2.00

**Water Demand based on Commercial landuse**

Land Use	Area (Ha.)	Average Daily Demand (Litres/min)	Maximum Hour (l/min)	Maximum Day (l/min)	Fire Flow Required (l/min)	Max Day + Fire Flow (l/min)
Development Area	0.67	13.0	35.8	26.0	12,000	12,026

## REQUIRED FIRE FLOW WORKSHEET - PROPOSED DEVELOPMENT

### Fire Underwriters Survey

**Project :** 350 Garfield Wright Boulevard  
**Project No:** 24015  
**Date:** 29-Aug-24

**Prepared by:** PM  
**Checked by:** PT  
**Last Revised:** 29-Aug-24

Guide for Determination of Required Fire Flow Copyright I.S.O

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

Where:

RFF = the Required Fire Flow in litres per minutes (LPM)  
 C = the Construction Coefficient is related to the type of construction of the building  
 A = the Total Effective Floor Area (effective building area) in square metres of the building

Type of Construction	Coefficient
Type V Wood Frame	1.5
Type IV-A Encapsulated Mass Timber	0.8
Type IV-B Rated Mass Timber	0.9
Type IV-C Ordinary Mass Timber	1.0
Type IV-D Un-Rated Mass Timber	1.5
Type III Ordinary	1.0
Type II Noncombustible	0.8
Type I Fire Resistive	0.6

Contents	Factor
NC Non-Combustible	-25%
LC Limited Combustible	-15%
C Combustible	0%
FB Free Burning	15%
RB Rapid Burning	25%

#### 1) Required Fire Flow

Type of Construction:

C=

A\*=

F=

Type III
1.0
918 m <sup>2</sup>
6,666 L/min

#### 2) Occupancy and Contents Adjustment Factor

Type of Occupancy

Contents Adjustment Factor

F=

C	0%	=	0 L/min
6666L/min +	0 L/min	=	6,666 L/min

#### 3) System Type Reduction

NFPA 13 Sprinkler:

Standard Water Supply:

Fully Supervised:

Total Credit

Reduction of:

F=

YES	30%
YES	10%
YES	10%
Total Credit	50%
Reduction of:	50% L/min = 3,333 L/min
F=	6666L/min - 3,333 L/min = 3,333 L/min

#### 4) Exposure Adjustment Charge

Building Face

North

East

South

West

Total

Building Face	Dist(m)	Charge
North	100	0%
East	100	0%
South	100	0%
West	100	0%
Total	0%	of 6665.7 L/min = 0 L/min

Separation Distance	Maximum Exposure Adjustment Charge
0 m to 3 m	25%
3.1 m to 10 m	20%
10.1 m to 20 m	15%
20.1 m to 30 m	10%
Greater than 30	0%

F= 3333L/min + 0L/min = 3,333 L/min

F=	3,000 L/min	(round to the nearest 1,000L/min)
F=	50 L/s	
F=	793 gpm	

**Min. Required Fireflow = 12,000 L/s per Town of East Gwillmbury Design Guidelines**

# APPENDIX B

**Sanitary Demand Calculations**

**Percolation Test Report**

**Septic System Design Information**

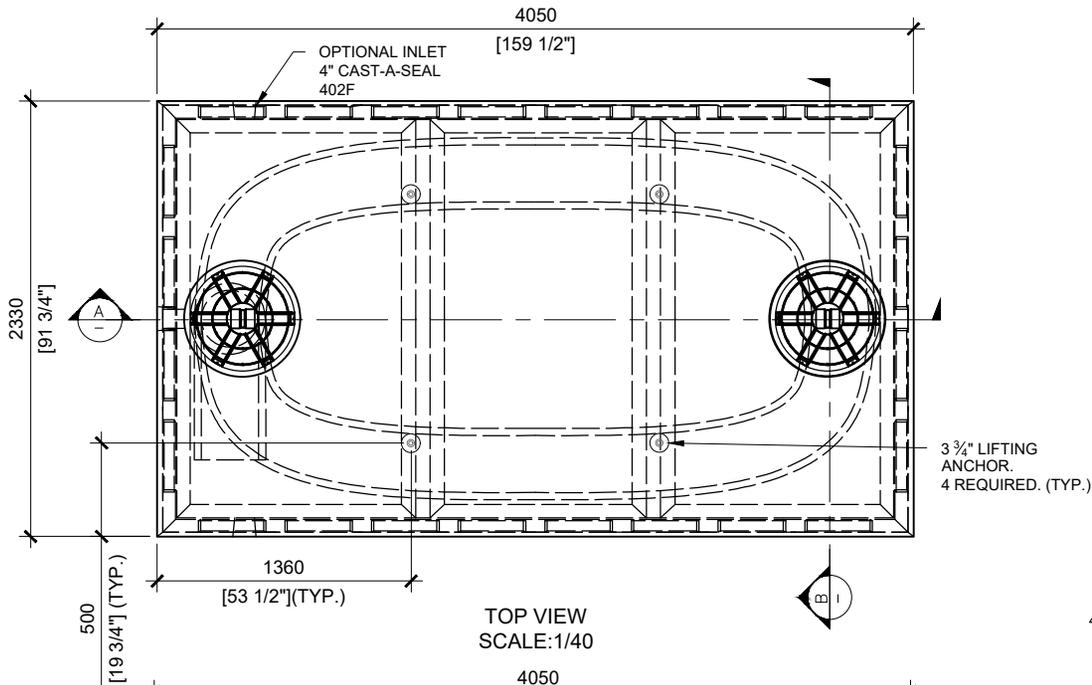
**Sanitary Design Calculations**

**Project:** 350 Garfield Wright Boulevard  
**Project No:** 24015  
**Client:** York Regional Police  
**Location:** East Gwillmbury, Ontario  
**Site Area:** 0.67 ha (development area only)  
**Date:** 29-Aug-24

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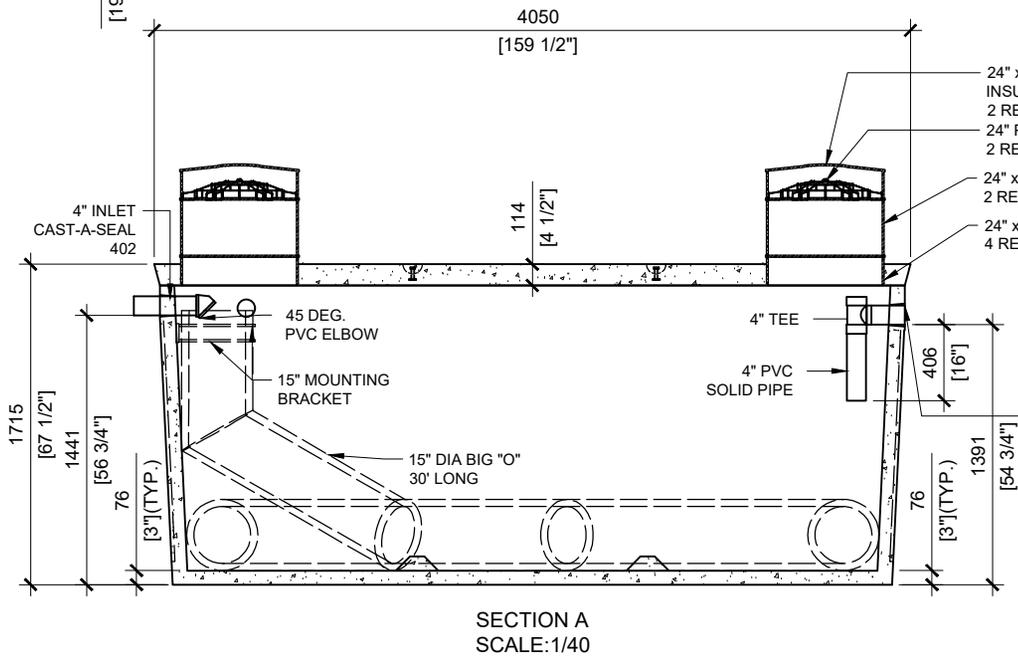
**Daily Sanitary Design Flow**

Ontario Building Code Non-Residential Design Flow Rates				
Occupancy	Unit	Daily Volume, Litres per unit *	Site Units	Daily Design Volume (Litres)
<b>Office Building</b>				
Per each 9.3 m <sup>2</sup> of floor space	9.3 sq.m	75	450	3,629
Per 2012 OBC Code, Table 8.2.1.3.B		Average Flow =		0.04 L/s
		=		2.52 L/min

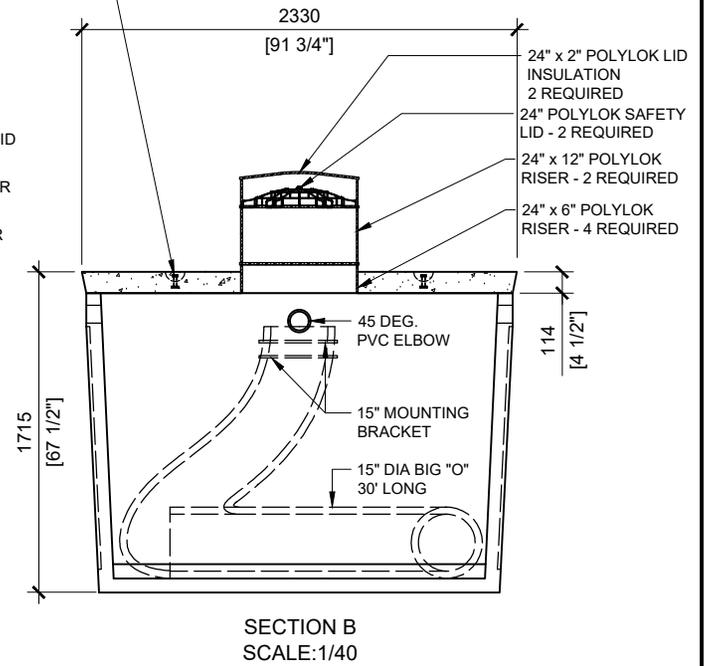


**GENERAL NOTES:**

1. UNITS ARE SEALED WITH BUTYL TAPE AT THE JOINTS
2. DELIVERY IS MADE BY CRANE-EQUIPPED TRUCKS
3. EXCAVATION MUST BE READY, SAFE AND ACCESSIBLE FOR UNLOADING FROM THE REAR OF THE TRUCK.
4. MIN OVERHEAD CLEARANCE OF 18FT (5.5 METERS) IS REQUIRED
5. ALL UNITS MUST BE HANDLED WITH PROPER LIFTING EQUIPMENT
6. MAXIMUM BURIAL DEPTH = 1 METRE IN FIRM SOIL AWAY FROM ANY VEHICULAR TRAFFIC
7. TUF-TITE SAFETY LIDS INSTALLED IN BOTH OPENINGS AS PER CSA-B66-21



3 3/4" LIFTING ANCHOR. 4 REQUIRED. (TYP.)



**MANUFACTURED:**  
LINDSAY, ON  
1-800-655-3430

**CONCRETE:** 35MPa/5000PSI  
**AIR ENTRAINMENT:** 5-8%  
**REINFORCEMENT:** STEEL TO CSA CAN  
A23.1 /A23.3 G30.18 Fy=400MPa

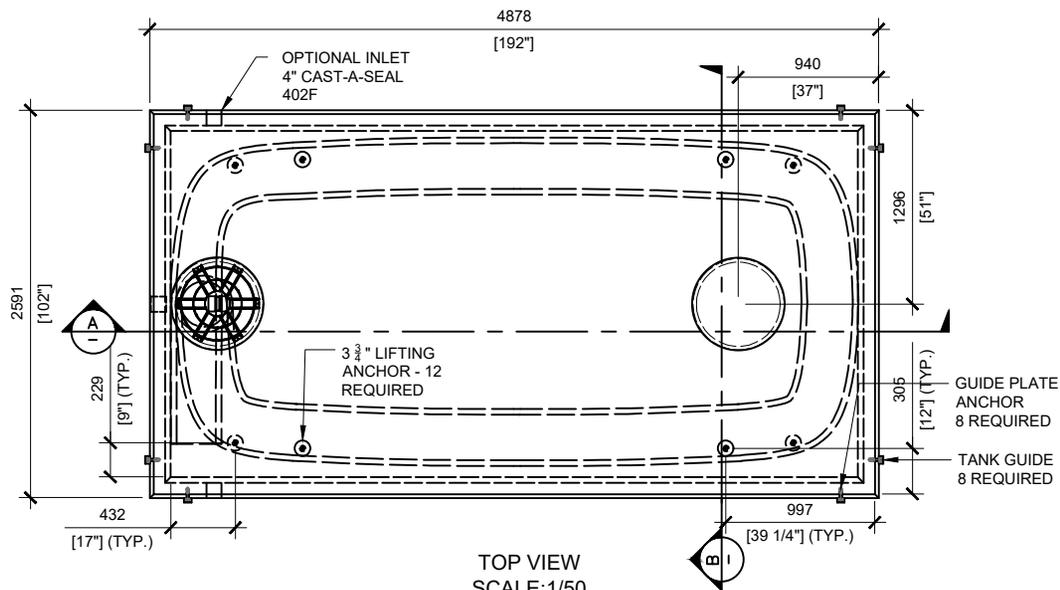
**WEIGHT:**  
21,380lbs / 9,676kg

**DRAWN BY:**  
PRASHAN

**DATE:**  
DEC/2023

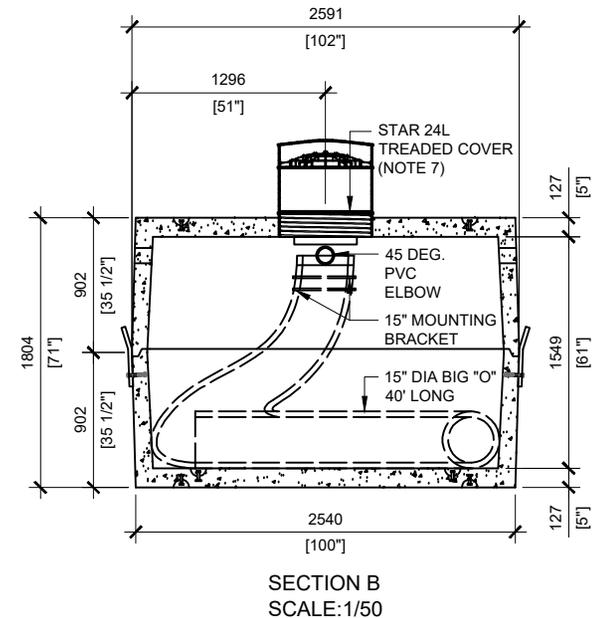
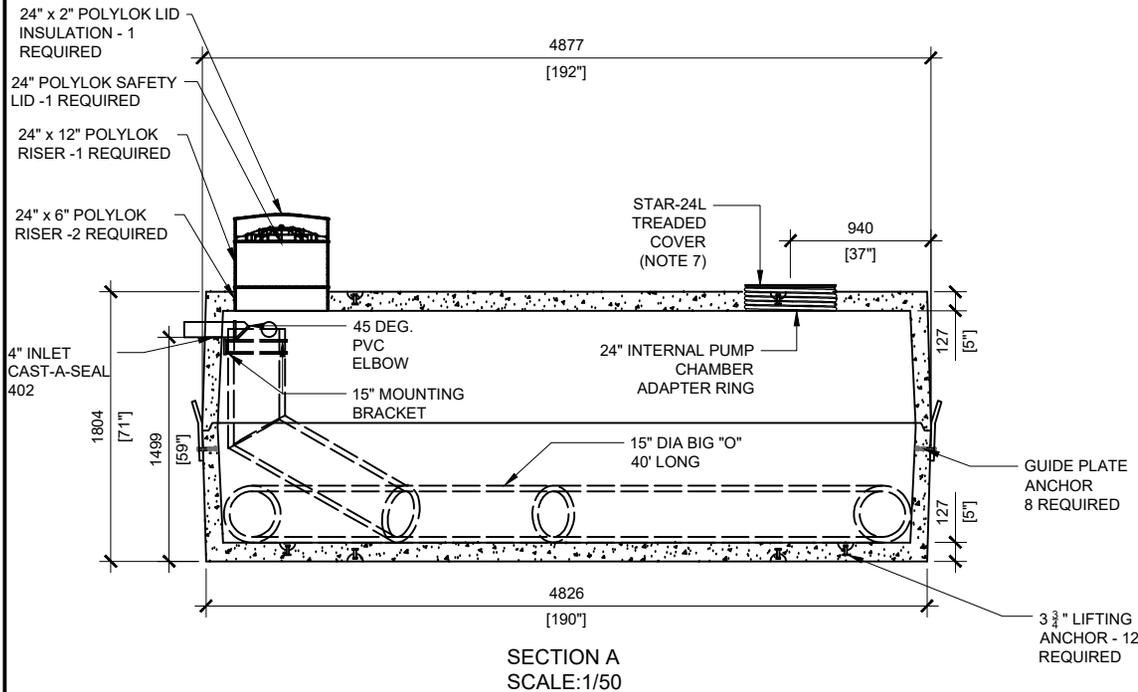
**WATERLOO AD-9000**

9,000 LITRES



**GENERAL NOTES:**

1. UNITS ARE SEALED WITH BUTYL TAPE AT THE JOINTS
2. DELIVERY IS MADE BY CRANE-EQUIPPED TRUCKS
3. EXCAVATION MUST BE READY, SAFE AND ACCESSIBLE FOR UNLOADING FROM THE REAR OF THE TRUCK.
4. MIN OVERHEAD CLEARANCE OF 18FT (5.5 METRES) IS REQUIRED
5. ALL UNITS MUST BE HANDLED WITH PROPER LIFTING EQUIPMENT (I.E. SPREADER BAR)
6. MAXIMUM BURIAL DEPTH = 1 METRE IN FIRM SOIL AWAY FROM ANY VEHICULAR TRAFFIC
7. THREADED COVER TO BE REMOVED AND REPLACED BY INTERNAL PUMP CHAMBER (PROVIDED BY WATERLOO) ON SITE



**MANUFACTURED:**  
LINDSAY, ON  
1-800-655-3430

**CONCRETE TYPE:** SCC  
**CONCRETE:** 45MPa at 28 days / 6,500PSI  
**AIR ENTRAINMENT:** 5-8%  
**REINFORCEMENT:** STEEL TO CSA CAN  
A23.1 / A23.3 G30.18 Fy=400MPa

**WEIGHT:**  
BOTTOM - 17,712lbs / 8,050kg  
TOP - 17,571lbs / 7,989kg

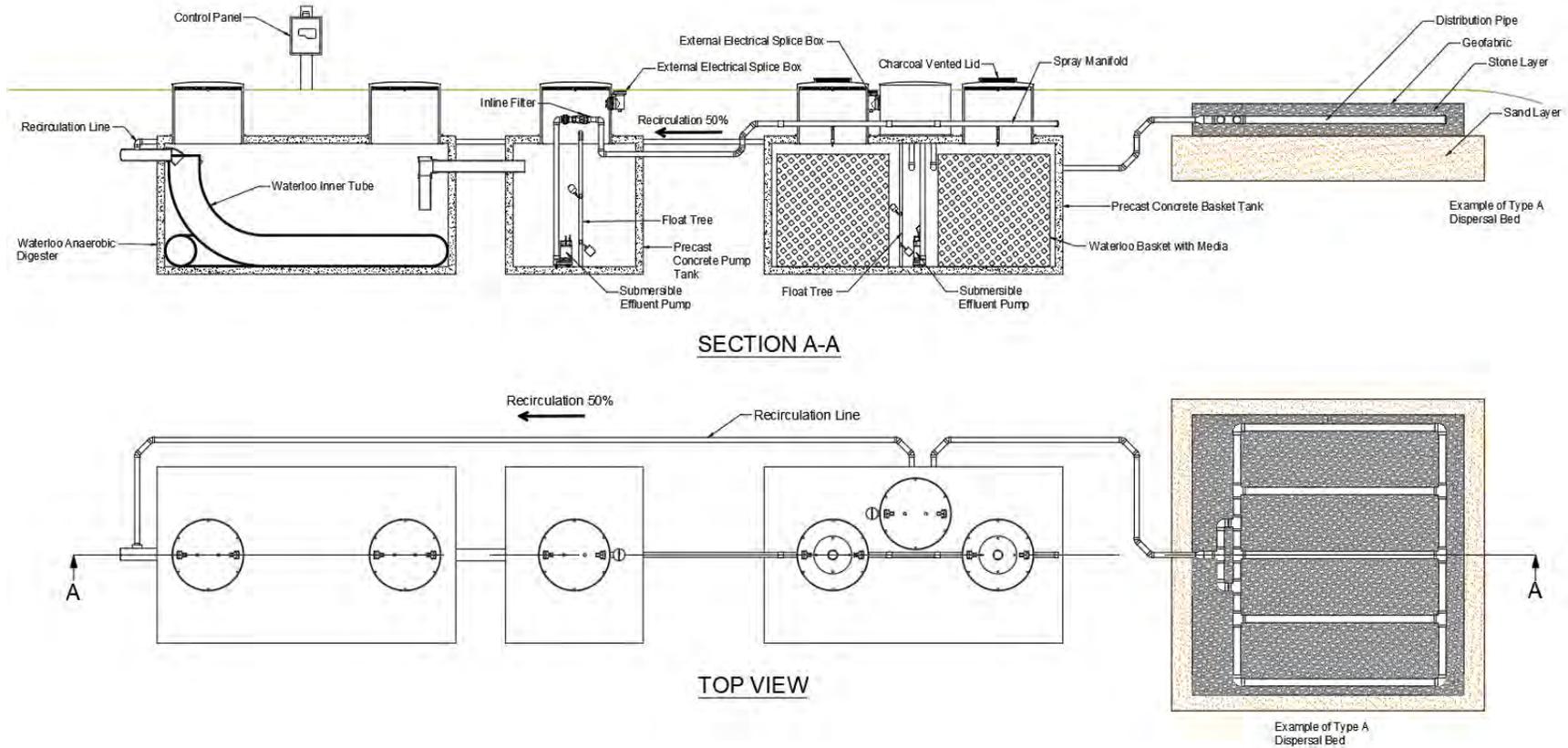
**DRAWN BY:**  
PRASHAN

**DATE:**  
DEC/2023

**WATERLOO ADIPC-14000**

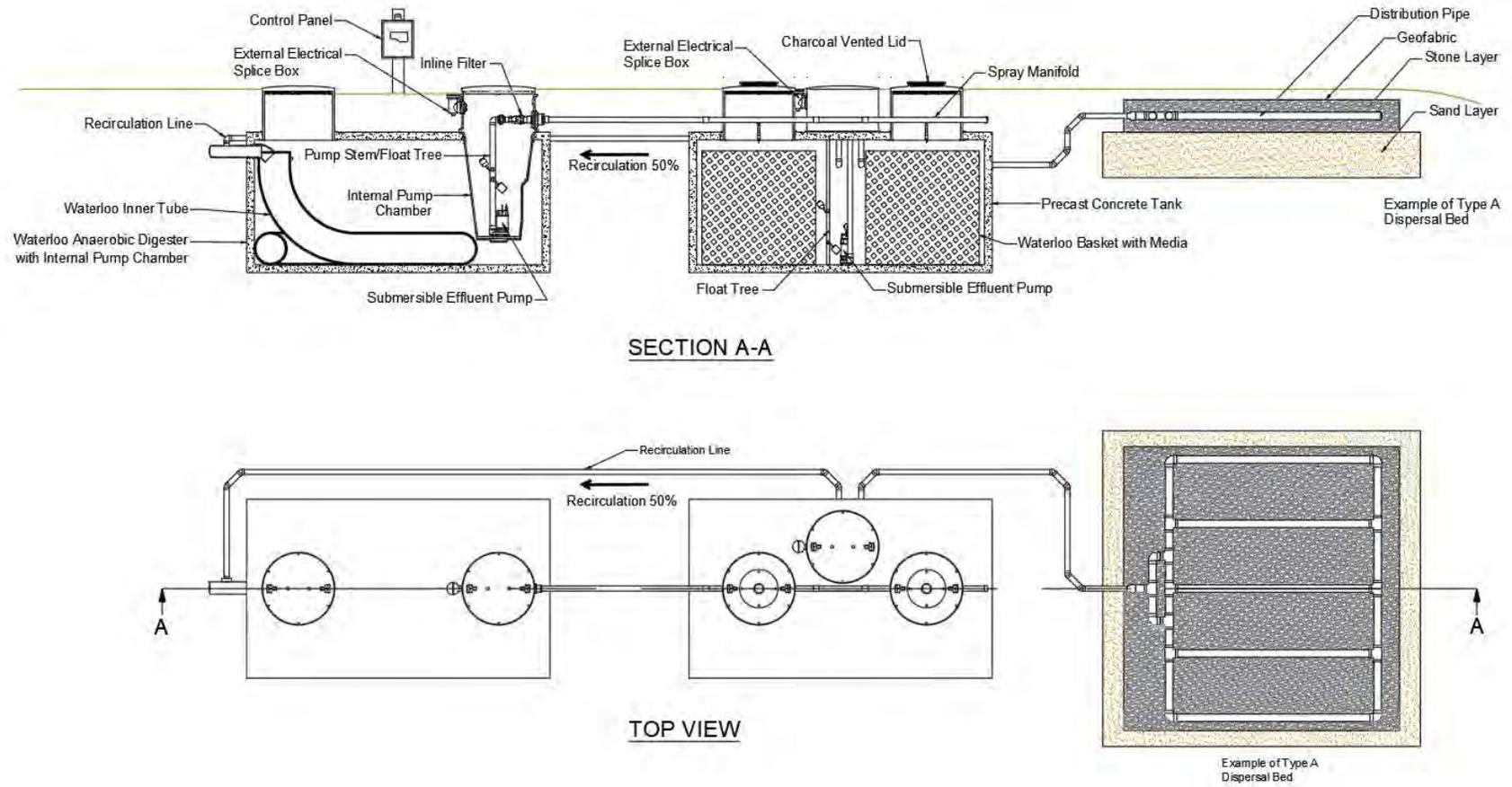
14,000 LITRES

## System Diagram - Baskets in Concrete Tank



**Figure 51.** Anaerobic digester, pump tank, and baskets in concrete tank system diagram

## System Diagram - Baskets in Concrete Tank



**Figure 50.** Anaerobic digester with internal pump chamber and baskets in concrete tank system diagram



July 31, 2024

Azimuth Environmental Consulting Inc.  
642 Welham Road  
Barrie, Ontario  
L4N 9A1

Attn: Brendan MacNaughton

**RE: Job No. 24-054**  
**Determination of Estimated T-Time**

---

GEI Consultants Ltd. (GEI) was provided with three (3) soil samples on July 23, 2024 to complete grain size analyses to determine the percolation rate of the tested soils (T-Time analysis).

The delivered samples were identified as shown below.

- TP-24-1-2, YRP Hanger
- TP-24-6-4, YRP Hanger
- TP-24-3-2, YRP Hanger

Three grain size distribution curves were developed by testing the above referenced soil samples in accordance with ASTM D6913 Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis and ASTM D7928 (sedimentation / hydrometer analysis). The result of the laboratory test and graphical representation of the grain size analyses are enclosed.

Determination of percolation rate is based on the “*Ministry of Municipal Affairs and Housing (MMAH) Supplementary Guidelines SB-6, Percolation Time and Soil Descriptions, September 14, 2012*”. Based on this document, a summary of the result and the estimated percolation rates of the soil are as follows:

Client Reference	Soil Description (MIT)	USCS Soil Classification	Coefficient of Permeability (K- cm/sec)	Estimated Percolation Rate or “T-Time” (mins/cm)
TP-24-1-2	SILT, Some Clay, Trace Sand	M.L.	$<10^{-6}$	>50 mins/cm
TP-24-6-4	SILT, Some Sand, Some Clay, Trace Gravel	M.L.	$10^{-6}$	50 mins/cm
TP-24-3-2	SANDY SILT, Some Clay, Trace Gravel	M.L.	$10^{-6}$	50 mins/cm

\*Reference MMAH Supplementary Standard SB-6, Table 2

It is noted that percolation time not only varies based on the grain size distribution but is also influenced by other soil characteristics such as the density of the soil, the structure of the soil, the percentage/mineralogy of clay, the plasticity of the soil, the organic content of the soil, and the groundwater table level which are not expressly calculated as part of a grain size analysis.

No field investigation was conducted by GEI in conjunction with the above testing and did not witness the depth or location in which these samples were obtained. GEI is providing the percolation rates as factual information, to be used in design by a qualified professional with due regard to the limitations as indicated above.

We trust this information is sufficient for your present purposes. Should you have any questions concerning the above, or if we can be of any further assistance, please do not hesitate to contact the undersigned.

Yours truly,  
**GEI Consultants Ltd.**



Donna Davidson-Gorry  
Laboratory Supervisor  
(705) 718-6604  
ddavidsongorry@geiconsultants.com



Andrew Jones  
Materials Testing and Inspection Practice Lead  
(705) 220-0060  
ajones@geiconsultants.com

Enclosures (3)

Grain Size Analysis (T-Time)

## **ENCLOSURE 1**

Grain Size Analysis (T-Time)



## **ENCLOSURE 2**

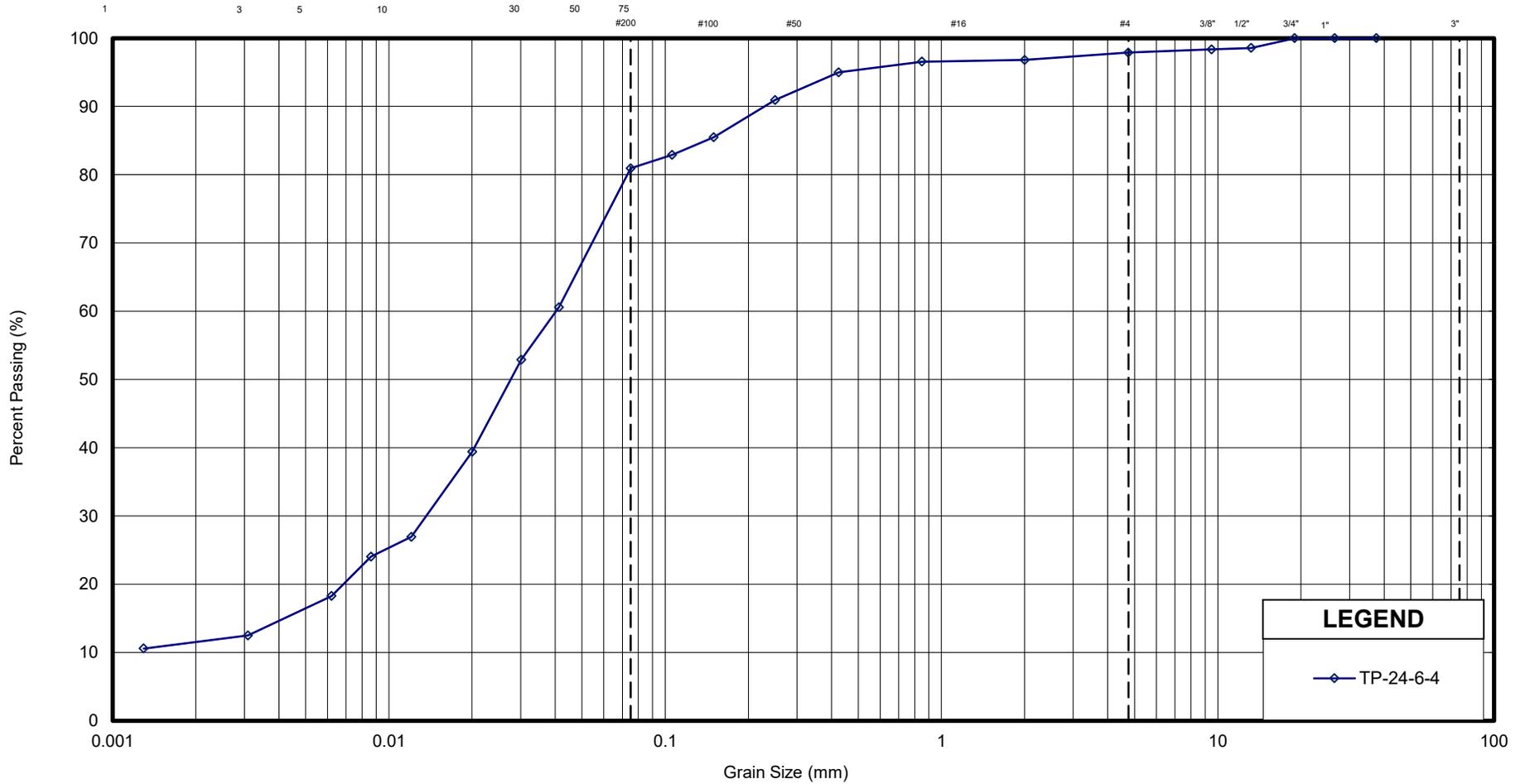
Grain Size Analysis (T-Time)

**UNIFIED SOIL CLASSIFICATION SYSTEM**

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (IMPERIAL)



GEI Lab No.	Description	Gr.	Sa.	Si.	Cl.	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>
7837	SILT, Some Sand, Some Clay, Trace Gravel	2	17	69	12	-	0.014	0.040	-	-



GRAIN SIZE DISTRIBUTION - Azimuth Environmental - YRP Hanger

**SILT**

FIGURE No.	
REF. No.	2005133
DATE	July 2024

## **ENCLOSURE 3**

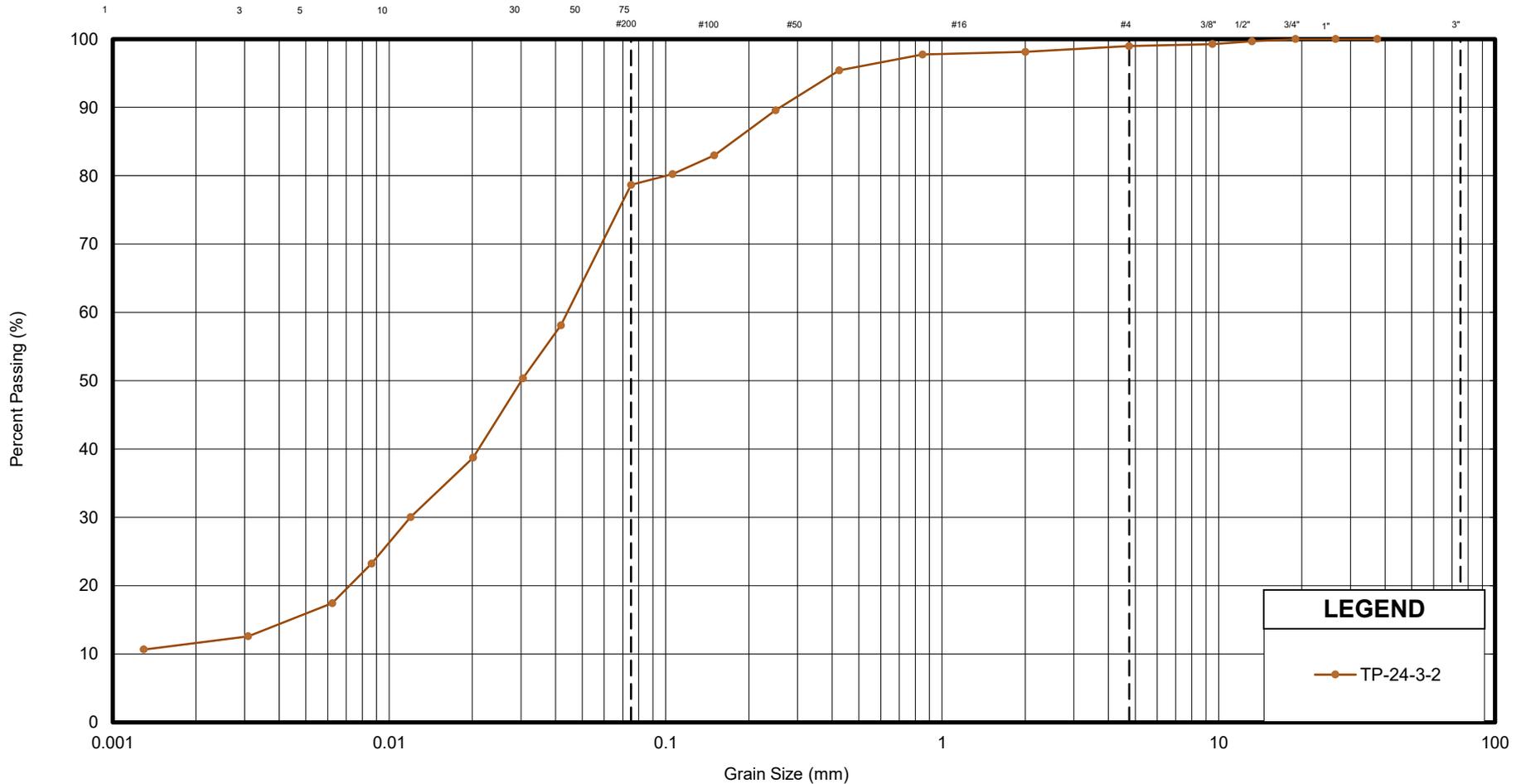
Grain Size Analysis (T-Time)

**UNIFIED SOIL CLASSIFICATION SYSTEM**

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (IMPERIAL)



LEGEND	
—●—	TP-24-3-2

GEI Lab No.	Description	Gr.	Sa.	Si.	Cl.	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>
7838	SANDY SILT, Some Clay, Trace Gravel	1	20	67	12	-	0.012	0.044	-	-



GRAIN SIZE DISTRIBUTION - Azimuth Environmental - YRP Hanger

**SANDY SILT**

FIGURE No.	
REF. No.	2005133
DATE	July 2024





# TEST PIT LOG

## Environmental Assessments & Approvals

<b>Project Name/ Project Client</b>	YRP Hanger/ York Regional Police	<b>Project Address</b>	90 Bales Drive East, Sharon, ON	<b>Date</b>	July 23, 2024
<b>Test Pit Number</b>	TP24-3	<b>Contractor</b>	Provided by Proponent	<b>Elevation</b>	NA
<b>Operator / Equipment</b>	Brock Excavation / Track Mounted Excavator	<b>Test Pit Size</b>	1m x 3m	<b>Datum</b>	Ground Surface
<b>Temperature</b>	25°C	<b>Weather</b>	Sunny	<b>Sample Type</b>	Soil

Depth		Soil description	Samples		pH	Remarks / Chemical Analysis
From (m)	To (m)		No.	Depth (mbgs)		
0.00	0.20	Brown, dry, loose sandy topsoil with organics and rootlets.	1	-	-	-
0.20	0.40	<i>Fill:</i> Light brown, dry, compact silt w/ fine sand and some stone and clay.	2	-	-	Sample submitted for grain size and T-time assessment.
0.40	0.50	<i>Buried Organics :</i> Dark brown to black, lots of organic material and woody debris.	3	-	-	-
0.50	1.95	<i>Fill:</i> Light brown, dry, compact to dense silt w/ some fine sand and clay. Some mottling after 50 cm. Becoming moist at 1.1 cm.	4	-	-	-
		<b>Test Pit Terminated at 1.95 mbgs</b>				

<b>Comments</b>	<b>Water Conditions in Test Pit</b>
Standpipe not installed in test pit prior to backfilling.	<input type="checkbox"/> Wet upon completion <input checked="" type="checkbox"/> Dry upon completion

**JOB No.** 24-054  
**TEST PIT No.** TP24-3  
**FIELD STAFF** B.Petterson





## TEST PIT LOG

### Environmental Assessments & Approvals

<b>Project Name/ Project Client</b>	YRP Hanger/ York Regional Police	<b>Project Address</b>	90 Bales Drive East, Sharon, ON	<b>Date</b>	July 23, 2024
<b>Test Pit Number</b>	TP24-6	<b>Contractor</b>	Provided by Proponent	<b>Elevation</b>	NA
<b>Operator / Equipment</b>	Brock Excavation / Track Mounted Excavator	<b>Test Pit Size</b>	1m x 3m	<b>Datum</b>	Ground Surface
<b>Temperature</b>	25°C	<b>Weather</b>	Sunny	<b>Sample Type</b>	Soil

Depth		Soil description	Samples		pH	Remarks / Chemical Analysis
From (m)	To (m)		No.	Depth (mbgs)		
0.00	0.20	Brown, dry, loose sandy topsoil with organics and rootlets.	1	-	-	-
0.20	0.96	<i>Fill:</i> Light brown, dry, compact silt w/ fine sand and some stone and clay.	2	-	-	-
0.96	1.60	<i>Fill:</i> Dark grey, moist silty clay w/ some organics and trace sand. Refuse present (i.e., wood debris, concrete, wire, plastic, etc.).	3	-	-	-
1.60	2.40	<i>Fill:</i> Light brown, dry, compact silt w/ some fine sand and clay; trace organics. Refuse present (i.e., wood, concrete, plastic, etc.). Pocket of medium-coarse sand at 45 cm.	4	-	-	Sample submitted for grain size and T-time assessment.
		<b>Test Pit Terminated at 2.1 mbgs</b>				
<b>Comments</b>			<b>Water Conditions in Test Pit</b>			
Standpipe not installed in test pit prior to backfilling.			<input type="checkbox"/> Wet upon completion <input checked="" type="checkbox"/> Dry upon completion			

**JOB No.** 24-054  
**TEST PIT No.** TP24-6  
**FIELD STAFF** B.Petterson

# YRP Hanger - Servicing Assessment

Test Pit Location Plan

## Legend

■ Test Pit Location



Google Earth

Image © 2024 Airbus

100 m



# APPENDIX C

**East Gwillimbury IDF Curve data and Post  
Development Stormwater Release Rates**

**MECP PBGT output**

**Jellyfish Filter ETV verification statement**

**Background YRIS SWMF design reports**

All storm sewers are to have a minimum horizontal separation of 2.5 m and a vertical clearance of 0.5 m from watermains in accordance with MOE regulations.

### **35.3 Termination Points**

All sewers shall be terminated at the subdivision limits when external drainage areas are considered in the design with suitable provision in the design of the terminal manholes to allow for the future extension of the sewer.

### **35.4 Sewer Alignment**

All storm sewers shall be laid in a straight line between manholes unless radial pipe has been designed as outlined in Section 36.9.

### **35.5 Pipe Crossings**

A minimum clearance of 75 mm shall be provided between the outside of the pipe barrel at the point of crossing for storm and sanitary sewers. A minimum clearance of 0.5 m shall be provided for all sewer and watermain crossings.

In the event the minimum clearances cannot be obtained, the designs must adhere to MOE policies. In addition the pipes shall be concrete encased to ensure that the pipes are properly bedded.

### **35.6 Changes in Pipe Size**

No decrease of pipe size from a larger upstream pipe to a smaller downstream size will be allowed due to the increase in grade.

### **35.7 Pipe Bedding and Backfill**

The class of pipe and the type of bedding shall be selected to suit loading and proposed construction conditions. Details and types of bedding and backfill are illustrated in OPSD 802.010 and 802.030. The width of the trench at the top of the pipe must be carefully controlled to ensure that the maximum trench width is not exceeded unless a higher class of bedding or higher pipe strength pipe is used. The recommendations of a Geotechnical Engineer will be required in determining strength of pipe required and construction methods to be used.

## **36.0 MANHOLES**

### **36.1 Location**

Manholes shall be constructed at the following locations:

- at changes in pipe size
- at pipe junctions
- at changes in pipe slope



## SWM DESIGN CALCULATIONS

### Post-Development Release Rate Calculations (Rational Method)

#### Area 100 - development area

**Project Name:** 350 Garfield Wright Boulevard  
**Municipality:** East Gwillmbury, Ontario  
**Project No.:** 24015  
**Date:** 28-Aug-24

**Prepared by:** PM  
**Checked by:** PT  
**Last Revised:** 28-Aug-24

#### Area 100

Location	<b>Burlington</b>
Area (ha)	<b>0.67</b>
Runoff Coefficient	<b>0.66</b>

#### Adjustment Factor

Up to 10-Year	<b>1.00</b>
25-Year	<b>1.10</b>
50-Year	<b>1.20</b>
100-Year	<b>1.25</b>

Event:	<b>2-Year</b>
a	648
b	4.000
c	0.784
Runoff Coefficient	<b>0.66</b>
AC	0.44
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>82</b>
Rational Flow Rate (l/s)	<b>100</b>

Event:	<b>10-Year</b>
a	1021
b	3.000
c	0.787
Runoff Coefficient	<b>0.66</b>
AC	0.44
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>136</b>
Rational Flow Rate (l/s)	<b>166</b>

Event:	<b>50-Year</b>
a	1488
b	3.000
c	0.803
Runoff Coefficient	<b>0.79</b>
AC	0.53
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>190</b>
Rational Flow Rate (l/s)	<b>279</b>

#### Rational Method

$$Q = KRCIA$$

Where:

- Q = Design flow (m<sup>3</sup> / sec)
- K = Conversion factor (0.00278)
- R = Return period factor
- C = Runoff coefficient
- I = Rainfall intensity (mm / hour)
- A = Contributing drainage area (ha)

Event:	<b>5-Year</b>
a	930
b	4
c	0.798
Runoff Coefficient	<b>0.66</b>
AC	0.44
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>113</b>
Rational Flow Rate (l/s)	<b>139</b>

Event:	<b>25-Year</b>
a	1100
b	2.000
c	0.776
Runoff Coefficient	<b>0.73</b>
AC	0.49
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>160</b>
Rational Flow Rate (l/s)	<b>216</b>

Event:	<b>100-Year</b>
a	1770
b	4.000
c	0.820
Runoff Coefficient	<b>0.83</b>
AC	0.55
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>203</b>
Rational Flow Rate (l/s)	<b>312</b>



## SWM DESIGN CALCULATIONS

### Post-Development Release Rate Calculations (Rational Method)

#### Area 101 - non development area

**Project Name:** 350 Garfield Wright Boulevard  
**Municipality:** East Gwillmbury, Ontario  
**Project No.:** 24015  
**Date:** 28-Aug-24

**Prepared by:** PM  
**Checked by:** PT  
**Last Revised:** 28-Aug-24

#### Area 100

Location	<b>Burlington</b>
Area (ha)	<b>0.27</b>
Runoff Coefficient	<b>0.25</b>

#### Adjustment Factor

Up to 10-Year	<b>1.00</b>
25-Year	<b>1.10</b>
50-Year	<b>1.20</b>
100-Year	<b>1.25</b>

Event:	<b>2-Year</b>
a	648
b	4.000
c	0.784
Runoff Coefficient	<b>0.25</b>
AC	0.07
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>82</b>
Rational Flow Rate (l/s)	<b>15.3</b>

Event:	<b>10-Year</b>
a	1021
b	3.000
c	0.787
Runoff Coefficient	<b>0.25</b>
AC	0.07
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>136</b>
Rational Flow Rate (l/s)	<b>25.4</b>

Event:	<b>50-Year</b>
a	1488
b	5.000
c	0.761
Runoff Coefficient	<b>0.30</b>
AC	0.08
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>189</b>
Rational Flow Rate (l/s)	<b>42.6</b>

#### Rational Method

$$Q = KRCIA$$

Where:

- Q = Design flow (m<sup>3</sup> / sec)
- K = Conversion factor (0.00278)
- R = Return period factor
- C = Runoff coefficient
- I = Rainfall intensity (mm / hour)
- A = Contributing drainage area (ha)

Event:	<b>5-Year</b>
a	930
b	4
c	0.798
Runoff Coefficient	<b>0.25</b>
AC	0.07
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>113</b>
Rational Flow Rate (l/s)	<b>21.2</b>

Event:	<b>25-Year</b>
a	1100
b	2.000
c	0.776
Runoff Coefficient	<b>0.28</b>
AC	0.07
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>160</b>
Rational Flow Rate (l/s)	<b>33.0</b>

Event:	<b>100-Year</b>
a	1770
b	5.000
c	0.761
Runoff Coefficient	<b>0.31</b>
AC	0.08
Tc (min)	<b>10</b>
Rainfall Intensity (mm/hr)	<b>225</b>
Rational Flow Rate (l/s)	<b>52.8</b>

## Project DEVELOPMENT Summary

**DEVELOPMENT: Parkin YRP**  
**Subwatershed: Black River**

Total Pre-Development Area (ha):	<b>0.6691</b>	Total Pre-Development Phosphorus Load (kg/yr):	<b>0.05</b>
----------------------------------	---------------	--	-------------

Pre-Development Land Use	Area (ha)	P coeff. (kg/ha)	P Load (kg/yr)
Hay-Pasture	0.6691	0.08	0.05

### POST-DEVELOPMENT LOAD

Post-Development Land Use	Area (ha)	P coeff. (kg/ha)	Best Management Practice applied with P Removal Efficiency	P Load (kg/yr)
High Intensity - Comm/Industrial	0.6691	1.82	Other 77%	0.28

*Jelly fish unit used to treat the site. The jelly fish unit is credited for 77% phosphorous removal. ETV certification is provided in SWM Report.*

Post-Development Area Altered:	0.67	P Load (kg/yr)
Total Pre-Development Area:	0.67	
Unaffected Area:	0	
Pre-Development:		0.05
Post-Development:		1.22
Change (Pre - Post):		-1.16
<b>2175% Net Increase in Load</b>		
Post-Development (with BMPs):		0.28
Change (Pre - Post):		-0.23
<b>423.25% Net Increase in Load</b>		

**DEVELOPMENT: Parkin YRP**  
**Subwatershed: Black River**

**CONSTRUCTION PHASE LOAD**

	<b>P Load (kg/yr)</b>
<b>SUMMARY WITH IMPLEMENTATION OF BMPs</b>	
Pre-Development:	<b>0.05</b>
Construction Phase Amortized Over 8 Years :	to be determined
Post-Development:	<b>0.28</b>
Post-Development + Amortized Construction:	<b>to be determined</b>
<b>Pre-Development Load - Post-Development Load:</b>	<b>-0.23</b>
<b>Conclusion:</b>	<b>423% Increase in Load</b>
<b>Pre-Development Load - (Post-Development + Amortized Construction Load):</b>	<b>to be determined</b>
<b>Conclusion:</b>	<b>to be determined</b>
<b>Based on a comparison of Pre-Development and Post-Development loads, and in consideration of Construction Phase loads, the Ministry would encourage the Municipality to:</b>	

# VERIFICATION STATEMENT

## GLOBE Performance Solutions

Verifies the performance of

### Jellyfish® Filter

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

Registration: GPS-ETV\_V2022-03-01

In accordance with

### ISO 14034:2016

**Environmental Management —  
Environmental Technology Verification (ETV)**



John D. Wiebe, PhD  
Executive Chairman  
GLOBE Performance Solutions

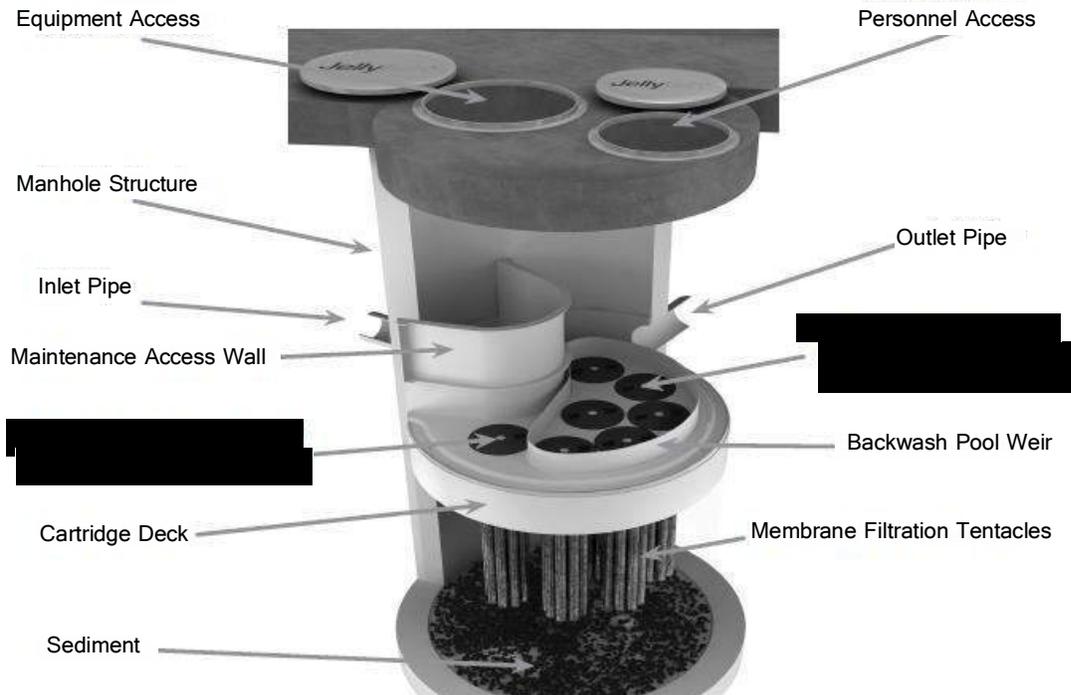
March 1, 2022  
Vancouver, BC, Canada



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

## Technology description and application

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



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

**Figure 1** depicts a cut-away graphic of a typical 6-ft diameter Jellyfish® Filter manhole with 6 hi-flo cartridges and 1 draindown cartridge (JF6-6-1). Stormwater influent enters the system through the inlet pipe and builds a pond behind the maintenance access wall, with the pond elevation providing driving head. Flow is channeled downward into the lower chamber beneath the cartridge deck. A flexible separator skirt surrounds the filtration zone where the filtration tentacles of each cartridge are suspended, and the volume between the vessel wall and the outside surface of the separator skirt comprises a pre-treatment channel. As flow spreads throughout the pre-treatment channel, floatable pollutants accumulate at the surface of the pond behind the maintenance access wall and also beneath the cartridge deck in the pre-treatment channel, while coarse sediments settle to the sump. Flow proceeds under the separator skirt and upward into the filtration zone, entering each filtration tentacle and depositing fine suspended sediment and associated particulate-bound pollutants on the outside surface of the membranes. Filtered water proceeds up the center tube of each tentacle, with the flow from each tentacle combining under the cartridge lid, and discharging to the top of the cartridge deck through the cartridge lid orifice. Filtered effluent from the hi-flo cartridges enters a pool enclosed by a 15-cm high weir, and if storm intensity and resultant driving head is sufficient, filtered water overflows the weir and proceeds across the cartridge deck to the outlet pipe. Filtered effluent discharging from the draindown cartridge(s) passes directly to the outlet pipe, and requires only a minimal amount of driving head (2.5 cm) to provide forward flow. As

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

## Performance conditions

The data and results published in this Verification Statement were obtained from the field testing conducted on a Jellyfish Filter JF6-6-1 (6-ft diameter manhole with 6 hi-flo cartridges and 1 draindown cartridge), in accordance with the requirements outlined by the Technical Guidance Manual for Evaluating Emerging Stormwater Treatment Technologies Technology Assessment Protocol – Ecology (TAPE) as written by the Washington State Department of Ecology, (WADOE, 2011). The drainage area providing stormwater runoff to the test unit was 86 acres and was 32% impervious. Throughout the monitoring period (March 2017 – April 2020), a total of 25 individual storm events were sampled. The Basic Treatment standard outlined in the TAPE requires ≥ 80% total suspended solids (TSS) removal at influent TSS concentrations ranging from 100 to 200 mg/L. In addition, the Phosphorus Treatment standard outlined in the TAPE requires ≥ 50% removal of total phosphorus (TP) at influent concentrations ranging from 0.10 to 0.5 mg/L. For this verification, the performance claim for TSS removal is for influent TSS concentration ≥ 100 mg/L, and the performance claim for TP removal is for influent TP concentration ≥ 0.1 mg/L. Based on these requirements, 15 and 18 sample pairs deemed qualified for evaluating the removal performance of TSS and TP, respectively. Prior to starting the performance testing program, a quality assurance project plan (QAPP) was submitted to and approved by the State of Washington Department of Ecology.

**Table 1** shows the specified and achieved TAPE criteria for storm selection and sampling.

**Table 1. Specified and achieved TAPE criteria for storm selection and sampling**

Description	TAPE criteria value	Achieved value
Total rainfall	> 3.8 mm (0.15 in)	> 3.8 mm (0.15 in) <sup>1</sup>
Minimum inter-event period	6 hours	6 hours
Minimum flow-weighted composite sample storm coverage	Minimum 70% including as much of the first 20% of the storm	> 70%
Minimum influent/effluent samples	10, but a minimum of 5 subsamples for composite samples	10, except for two events that had 9 aliquots
Total sampled rainfall	N/A	8.29 in
Number of storms	Minimum 15 (preferably 20)	25

<sup>1</sup>N.B. Storm event depth was greater than the TAPE rainfall depth guideline of 0.15 inches for all events sampled, except for the 3/21/2017, 3/22/2019, 3/26/2019, and 04/13/2019 events. Given the size of the drainage basin, storm events below this threshold produced adequate runoff volume for sampling. Only two of these events were used to evaluate performance, and all had rainfall depths of 0.11 inches or greater. These events were included as their runoff volumes, precipitation durations, and influent TSS concentrations were all within range of the total data set.

The 6-ft diameter test unit has sedimentation surface area of 2.62 m<sup>2</sup> (28.26 ft<sup>2</sup>). Each of the seven filter cartridges employed in the test unit uses filtration tentacles of 137 cm (54 in) length, with filter surface area of 35.4 m<sup>2</sup> (381 ft<sup>2</sup>) per cartridge, and total filter surface area of 247.8 m<sup>2</sup> (2667 ft<sup>2</sup>) for the seven cartridges combined. The design treatment flow rate is 5 L/s (80 gal/min) for each of the six hi-flo

cartridges and 2.5 L/s (40 gal/min) for the single draindown cartridge, for a total design treatment flow rate of 32.5 L/s (520 gal/min) at design driving head of 457 mm (18 in). This translates to a filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m<sup>2</sup> (0.21 gal/min/ft<sup>2</sup>) for each hi-flo cartridge and 0.07 L/s/m<sup>2</sup> (0.11 gal/min/ft<sup>2</sup>) for the draindown cartridge. The design flow rate for each cartridge is controlled by the sizing of the orifice in the cartridge lid. The distance from the bottom of the filtration tentacles to the sump is 61 cm (24 in).

## Performance claim(s)

The Jellyfish® Filter demonstrated the removal efficiencies indicated in **Table 2** for TSS and TP during field monitoring conducted in accordance with the Washington State Department of Ecology’s Technology Assessment Protocol – Ecology (TAPE), and using the following design parameters:

- System hydraulic loading rate (system treatment flow rate per unit of sedimentation surface area) of 12.5 L/s/m<sup>2</sup> (18.4 gal/min/ft<sup>2</sup>) or lower
- Filtration flux rate (flow rate per unit filter surface area) of 0.14 L/s/m<sup>2</sup> (0.21 gal/min/ft<sup>2</sup>) or lower for each hi-flo cartridge and 0.07 L/s/m<sup>2</sup> (0.11 gal/min/ft<sup>2</sup>) or lower for each draindown cartridge
- Distance from the bottom of the filtration tentacles to the sump of 61 cm (24 in) or greater
- Driving head of 457 mm (18 in) or greater

**Table 2. Bootstrapped mean, median, and 95% confidence interval (median) for removal efficiencies of Total Suspended Solids (TSS) and Total Phosphorus (TP)**

Parameter	Mean (%)	Median (%)	Median – 95% Lower Limit	Median – 95% Upper Limit
TSS <sup>1</sup>	87.6	90.1	85.1	91.6
TP <sup>2</sup>	77.3	77.5	70.8	85.6

<sup>1</sup> TSS influent concentration ≥ 100 mg/L

<sup>2</sup> TP influent concentration ≥ 0.1 mg/L

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

The performance claims can be applied to other Jellyfish® Filter models smaller or larger than the tested model as long as the untested models are designed in accordance with the design parameters specified in the performance claims.

## Performance results

### Performance Claims – Removal Efficiency for Total Suspended Solids

Raw data summarizing the percent removal of total suspended solids (TSS) by the Jellyfish® Filter at the design system hydraulic loading rate of 12.5 L/s/m<sup>2</sup> (18.4 gal/min/ft<sup>2</sup>) for 15 sample pairs deemed qualified are presented in **Table 3**. Data were analyzed and evaluated using a bootstrap approach of random sampling by replacement to estimate population distribution and thereby the upper and lower limit of the confidence interval.

**Table 3. Raw data summarizing the percent removal of total suspended solids (TSS)**

Event ID	TSS Influent (mg/L)	TSS Effluent (mg/L)	TSS Removal (%) (Inf ≥ 100 mg/L)
3/21/2017	102.0	22.0	78.4
4/7/2017	201.0	30.8	84.7
4/12/2017	108.0	24.4	77.4
4/19/2017	452.0	44.6	90.1
4/26/2017	257.0	10.0	96.1

6/15/2017	134.0	10.4	92.2
3/8/2018	755.0	47.2	93.8
3/14/2018	181.0	27.0	85.1
3/22/2018	224.0	20.0	91.1
4/5/2019	171.0	23.0	86.6
4/13/2019	117.0	25.0	78.6
5/18/2019	254.0	20.0	92.1
12/7/2019	200.0	17.0	91.5
3/30/2020	605.0	51.0	91.6
4/20/2020	210.0	29.0	86.2
<b>n</b>	15	15	15
<b>Min</b>	102.0	10.0	77.4
<b>Max</b>	755.0	51.0	96.1
<b>Median</b>	201.0	24.4	90.1
<b>Mean</b>	264.7	26.8	87.7
<b>SD</b>	190.9	12.3	5.9

**Performance Claims – Removal Efficiency for Total Phosphorus**

Raw data summarizing the percent removal of total phosphorus (TP) by the Jellyfish® Filter at the design system hydraulic loading rate of 12.5 L/s/m<sup>2</sup> (18.4 gal/min/ft<sup>2</sup>) for 18 sample pairs deemed qualified are presented in **Table 4**. Data were analyzed and evaluated using a bootstrap approach of random sampling by replacement to estimate population distribution and thereby the upper and lower limit of the confidence interval.

**Table 4. Raw data summarizing the percent removal of total phosphorus (TP)**

<b>Event ID</b>	<b>TP Influent (mg/L)</b>	<b>TP Effluent (mg/L)</b>	<b>TP Removal (%) (Inf ≥ 0.1 mg/L)</b>
4/7/2017	0.706	0.092	87.0
4/12/2017	0.338	0.076	77.5
4/19/2017	0.500	0.036	92.8
4/26/2017	0.504	0.042	91.7
5/13/2017	0.256	0.110	57.0
6/8/2017	0.256	0.104	59.4
6/15/2017	0.362	0.052	85.6
3/8/2018	1.75	0.130	92.6
3/14/2018	0.652	0.094	85.6
3/22/2018	0.364	0.072	80.2
3/27/2019	0.226	0.070	69.1
4/5/2019	0.337	0.092	72.9
4/13/2019	0.249	0.087	65.1
5/18/2019	1.09	0.173	84.1
12/7/2019	0.335	0.105	68.7
12/19/2019	0.211	0.093	56.2
3/30/2020	1.05	0.092	91.2
4/20/2020	0.451	0.112	75.2
<b>n</b>	18	18	18
<b>Min</b>	0.211	0.036	56.2
<b>Max</b>	1.75	0.173	92.8
<b>Median</b>	0.363	0.092	78.9
<b>Mean</b>	0.535	0.091	77.3
<b>SD</b>	0.400	0.032	12.5

## Verification

The verification was completed by the Verification Expert, the Centre for Advancement of Water and Wastewater Technologies (“CAWT”), contracted by GLOBE Performance Solutions, using the International Standard **ISO 14034:2016 Environmental management – Environmental technology verification (ETV)**. Data and information provided by Imbrium Systems to support the performance claim included the performance monitoring report “General Use Level Designation Technical Evaluation Report” prepared by CONTECH Engineered Solutions, Portland, OR, USA, and dated December 28, 2020. This report is based on a field testing completed by CONTECH personnel at a site in Dundee, Oregon between March 2017 and April 2020 in accordance with the Technical Guidance Manual for Evaluating Emerging Stormwater Treatment Technologies Technology Assessment Protocol – Ecology (TAPE) as written by the Washington State Department of Ecology (WADOE, 2011).

## What is ISO 14034:2016 Environmental management – Environmental technology verification (ETV)?

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

**For more information on the Jellyfish® Filter please contact:**

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info@imbriumsystems.com

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

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Vancouver, BC  
V6C 3E2, Canada  
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etv@globepformance.com  
www.globepformance.com

### **Limitation of verification - Registration: GPS-ETV\_V2022-03-01**

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

Prepared for the *Regional Municipality of York*

## Technical Design Brief

Stormwater Management Facilities

York Region Industrial Subdivision 19T-94016

Town of East Gwillimbury, Regional Municipality of York

**Project: 5390**

**July, 2004**

consulting engineering | planning | environmental approvals



Town of  
**East Gwillimbury**

**Community Programs &  
Infrastructure**

**Don Allan, CET, CST**  
*Manager, Development Engineering*

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Direct: 905-478-3819 Fax: 905-478-8545  
dallan@eastgwillimbury.ca

**Technical Design Brief, Stormwater Management Facilities  
York Region Industrial Subdivision 19T-94016, Part of Lot 2, Concession 4  
Northeast of Woodbine Avenue and Davis Drive  
Town of East Gwillimbury, Regional Municipality of York**

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**Technical Design Brief, Stormwater Management Facilities**  
**York Region Industrial Subdivision 19T-94016, Part of Lot 2, Concession 4**  
**Northeast of Woodbine Avenue and Davis Drive**  
**Town of East Gwillimbury, Regional Municipality of York**

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## 1.0 INTRODUCTION

Cumming Cockburn Limited has been retained by the Regional Municipality of York to develop the overall stormwater management plan for the York Region's Industrial Subdivision (19T-94016), located northeast of the intersection of Woodbine Avenue and Davis Drive at the Town of East Gwillimbury within the Regional Municipality of York (Region) as illustrated in **Figure 1**.



**Figure 1. Site Location**

As shown in **Figure 2**, the subject property consists of five parcels of land (Blocks 1, 2, 3, 4 and 5) with a total area over 60 ha. The Black River transverses the eastern portion (within Block 5) of the site and naturally collects and drains runoff from Blocks 4 and 5, and the east half of Block 3. An intermittent swale transverses the central portion (within Block 3) of the site and naturally collects and drains runoff from Blocks 1 and 2, the west half of Block 3 and the external industrial development area to the south.

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**YORK REGION INDUSTRIAL SUBDIVISION**  
 Town of East Gwillimbury, Regional Municipality of York

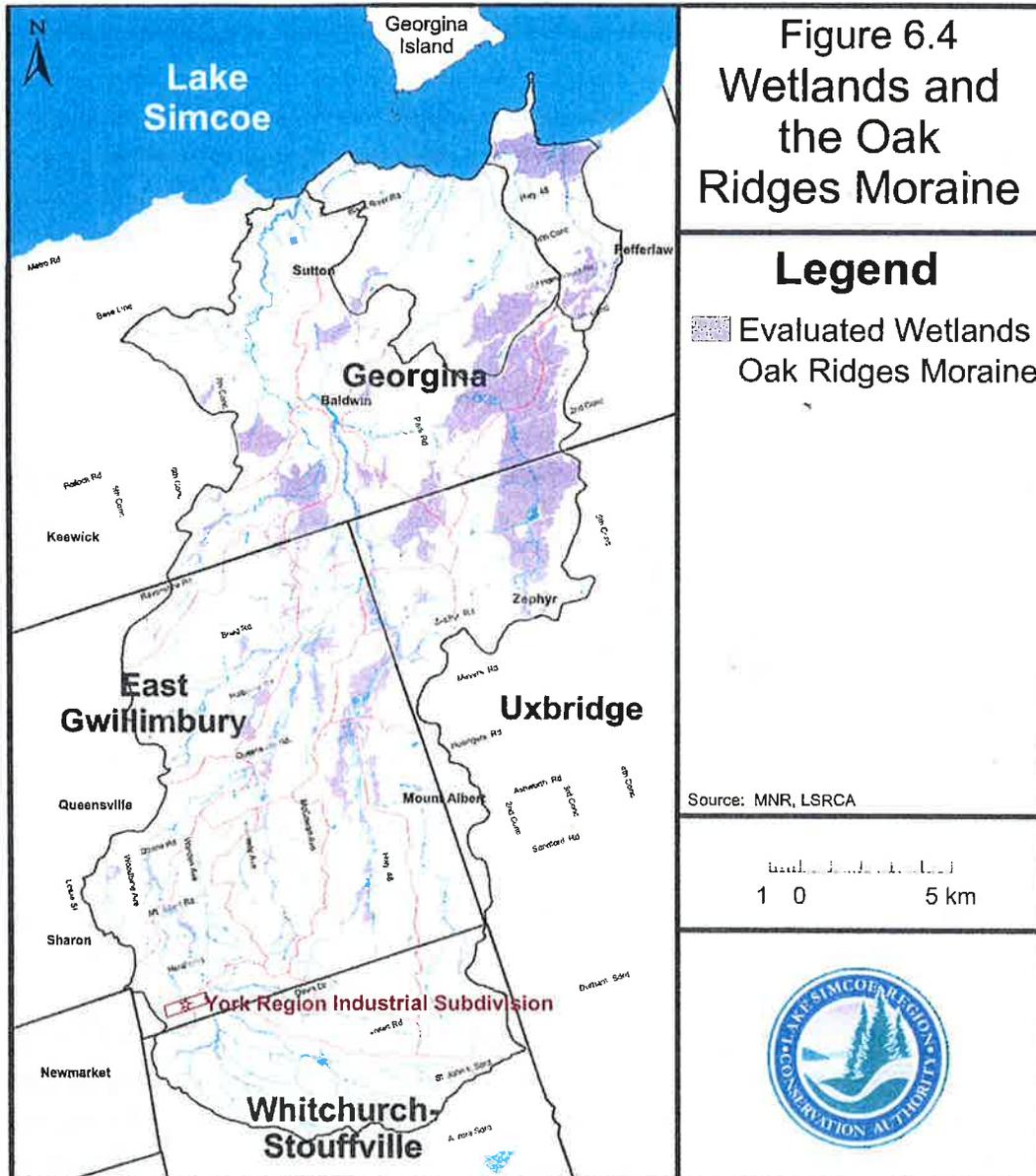
**FIGURE 2 EXISTING LAND USE**



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SCALE 1:5000

According to the draft State of the Watershed Report for Black River Subwatershed (referred to as the Black River Subwatershed Study in this Report) by the Lake Simcoe Conservation Authority (LSRCA) in August 2002, the subject site and its external industrial development area to the south are located within the headwaters of the Black River and entirely outside of the designated Oak Ridges Moraine protection area as illustrated in **Figure 3**. As a result, it is not subject to the requirements of the Oak Ridges Moraine Conservation Act (Ont. Reg. 140/02).



**Figure 3. Site Location within Black River Watershed in Relationship with Oak Ridges Moraine (LSRCA, 2002)**

A Functional Servicing Report for the York Region Industrial Subdivision was completed by URS Cole Sherman & Associated Limited in August 2002, and proposed two end-of-pipe stormwater management facilities of SWMF1 and SWMF2 located within Blocks 2 and 4 respectively to treat and attenuate runoff from the developments in Blocks 1 and 2, and in Blocks 3 and 4 respectively, and Block 5 is designated as open space. In addition to these two facilities, Cumming Cockburn Ltd. proposed one additional facility SWMF3 located at the south half of the intermittent swale in Block 3 to provide *Enhanced* (Level 1) water quality, erosion and flood controls not only for a part of the York Region Industrial Subdivision, but also for the external existing and future industrial development areas to the south (including the areas associated with the extension and urbanization of Garfield Wright Boulevard) as desired by the Town.

Since the engineering design of SWMF1 has been completed by Marshall Macklin Monaghan Ltd. in Oct. 2003 and approved by the regulatory agencies to service the immediate development of the York Region Material Recovery and Transfer Facility in Block 2 and the future industrial development in Block 1, the technical design brief for the two remaining facilities SWMF2 and SWMF3 was prepared by Cumming Cockburn Limited in Nov. 2003 and submitted to the Town, the LSRCA, and the Regional Municipality of York for review and comments.

According to the review comments received from the LSRCA dated Feb. 2 and April 16 of 2004 (see **Appendix A**), the Authority doesn't support the on-line facility SWMF3 because fish (Brook Stickleback) were observed and captured at a culvert approximately 188 m downstream of Garfield Wright Boulevard, based on the finding of a site visit conducted by the Authority's fisheries biologist and aquatic ecologist on April 16, 2004. The intermittent swale is, therefore, classified by the LSRCA as the headwaters of a fully functioning cold to coolwater tributary that would not require stormwater treatment regardless the existing development of Bales Industrial Subdivision.

Considering the LSRCA's concern on the fishery issue and the immediate development needs associated with the urbanization and extension of Garfield Wright Blvd, Cumming Cockburn Ltd., after consultation with the LSRCA, the Town and the Region (see **Appendix A**), proposes to use SWMF2 (located at the southeast corner of Block 4) to accommodate all the developments within Block 4, east portion of Block 3, and the areas associated with the extension and urbanization of Garfield Wright Boulevard. The intent of the revised stormwater management plan is to maximize useable lands, minimize the number of the stormwater management facilities, and maintain the current storm drainage pattern to the intermittent swale as much as possible for the existing fish habitat.

Since only a small portion of Block 4 is subject to the immediate development in addition to the extension and urbanization of Garfield Wright Boulevard, the primary objective of this report is to provide the technical design brief for the interim SWMF2 to accommodate the immediate development needs (i. e. the interim development condition), but a separate easement block (Block 9) is reserved and designated based on the engineering design of the ultimate SWMF2 for future extension of the interim SWMF2 to accommodate the full industrial development (i. e. the ultimate development condition).

## 2.0 SITE DESCRIPTION AND EXISTING DRAINAGE CONDITIONS

### 2.1 Background Information

The Regional Municipality of York, Lake Simcoe Region Conservation Authority, Town of East Gwillimbury, Matrix Management Corp., URS Architects & Engineers Canada Inc., and Marshall Macklin Monaghan Limited were consulted to acquire the available background information and clarify the stormwater management design criteria. The primary background materials gathered during the preparation of this report can be summarized as follows:

- Draft Plan for the York Region Industrial Subdivision, URS Architects & Engineers Canada Inc., April. 2004.
- Stormwater Management Plan for the York Region Waste Transfer Station, Marshall Macklin Monaghan Limited, Oct. 2003.
- Geotechnical Investigation for Pavement Design and Storm Sewers along Bales Drive and Roads "A" and "B", Shaheen & Peaker Ltd., Sept. 26, 2003.
- Functional Servicing Report for the York Region Industrial Subdivision, URS Cole Sherman & Associated Ltd., Aug. 2002.
- Sub-Surface Soils Investigation for York Industrial Subdivision, Jagger Hims Ltd., Aug. 2002.
- State of the Watershed Report: Black River Subwatershed (Draft), LSRCA, August 2002.
- Preliminary Hydrogeological Study for the Proposed Integrated Solid Waste Processing and Transfer Facility, Gartner Lee Limited, Feb. 2002.
- Soil Survey of York County - Report No. 19 of the Ontario Soil Survey, Ontario Ministry of Agriculture and Food and the Research Branch of Agriculture Canada, March 1955.

### 2.2 Existing Land Use

The subject property is legally described as Part of Lot 2, Concession 4 within the Town of East Gwillimbury. The site is bounded by Woodbine Avenue to the west, agricultural lands to the north, existing residential subdivision and open space to the east, and Garfield Wright Boulevard (formerly Bales Drive) and agricultural lands to the south.

**Figure 2** presents the existing land use within and around the subject site. Except for an existing farmhouse with a driveway to Woodbine Ave. in Block 1, most of the subject lands are currently used for the sod farming. Block 5 (located at the east end of the subject site) lies mainly within the regulatory floodplain of the Black River and is characterized by natural riparian marsh meadow, typical of saturated soils with the groundwater table at or near to the ground surface.

To the south of the subject site, there is an existing industrial subdivision (referred to as Bales Industrial Subdivision in this Report) that is bounded by Garfield Wright Boulevard to the north,

Bales Drive East to the east, Davis Drive to the south and Bales Drive West to the west. Within the Bales Industrial Subdivision, there are several industrial buildings with large parking lots and vacant lands that can be developed in the future.

### **2.3 Native Soils and Groundwater Characteristics**

The Soil Survey of York County (Report 19 of the Ontario Soil Survey published by the Ontario Ministry of Agriculture and Food and the Research Branch of Agriculture Canada in March 1955) and the findings of several site geotechnical investigations were used to determine soil types, soil drainage characteristics and groundwater natures within and around the subject site.

It was found that Schomberg silt loam (Shs) is dominant underlying soil presented within Blocks 1, 2 and 3, and the external area to the south, in addition to Brighton sandy loam (BrsL) mainly presented within Blocks 4 and 5.

The parent materials of Schomberg silt loam contain lacustrine, grey and calcareous clay and silt clay. Schomberg silt loam belongs to the soil group of Grey-Brown Podzolic with good drainage characteristics, and it is classified as the hydrologic soil group of BC with the runoff curve number (CN) of 68 under the Level II (average) antecedent moisture conditions for pasture, open space, lawns and parks, based on the Drainage Management Manual published by the Ontario Ministry of Transportation in 1997.

The parent material of Brighton sandy loam has well sorted grey, calcareous sand and stratified sand and gravel. Brighton sandy loam belongs to the soil group of Grey-Brown Podzolic with very good drainage nature, and it is classified as the hydrologic soil group of AB with the runoff curve number (CN) of 50 under the Level II (average) antecedent moisture conditions for pasture, open space, lawns and parks.

There is a weathered zone (layer with enhanced permeability) extending between 3.0 and 5.0 m in depth that is host to fluctuating groundwater levels. As the area is underlain by low permeability soils, there is a high-perched groundwater table that seasonally fluctuates between 0.5 m and 2.0 m below the ground surface, almost across the entire site. The depth of the groundwater table appears to become shallower towards the east when approaching the main branch of the Black River in Block 5.

A very minor portion of groundwater flows towards the Black River through the weathered zone, and forms a shallow horizontal groundwater flow from west to east. The results of geotechnical investigations indicate that the bulk of groundwater moves downwards since the average annual horizontal groundwater flow is only 1.1 litres/minute for the full width of the site, comparing to the downward flow rate of 47 litres/minute. Therefore, the water that soaks into the ground largely

moves downward to recharge the regional groundwater system, and only a very small part of it (<3%) moves laterally toward the Black River via the weathered zone. The hydrogeological study by Gartner Lee Ltd. in February 2002 indicated that this perched groundwater has low ecological value. All geotechnical reports to date further suggest that groundwater will not be a significant excavation issue and may be handled by a conventional sump or pumping technique.

## 2.4 Existing Drainage and Environmental Features

**Figure 4** shows existing drainage conditions within and around the subject site. Currently, runoff from Blocks 4 and 5, and the east half of Block 3 drains easterly towards the main branch of the Black River. Runoff from Blocks 1 and 2, west half of Block 3 and the existing and future industrial development areas to the south drains towards the intermittent swale. The finding and conclusion of the geotechnical investigation reports indicate that the north portion of the intermittent swale is a localized shallow groundwater discharge zone during wet periods of a year. In addition, it also services as a surface drainage channel. According to the finding of a site visit by the LSRCA's fisheries biologist and aquatic ecologist on Apr. 16, 2004, the swale is classified as the headwater of a fully functioning cold to coolwater tributary that would not require stormwater treatment regardless the existing development of Bales Industrial Subdivision (see **Appendix A**).

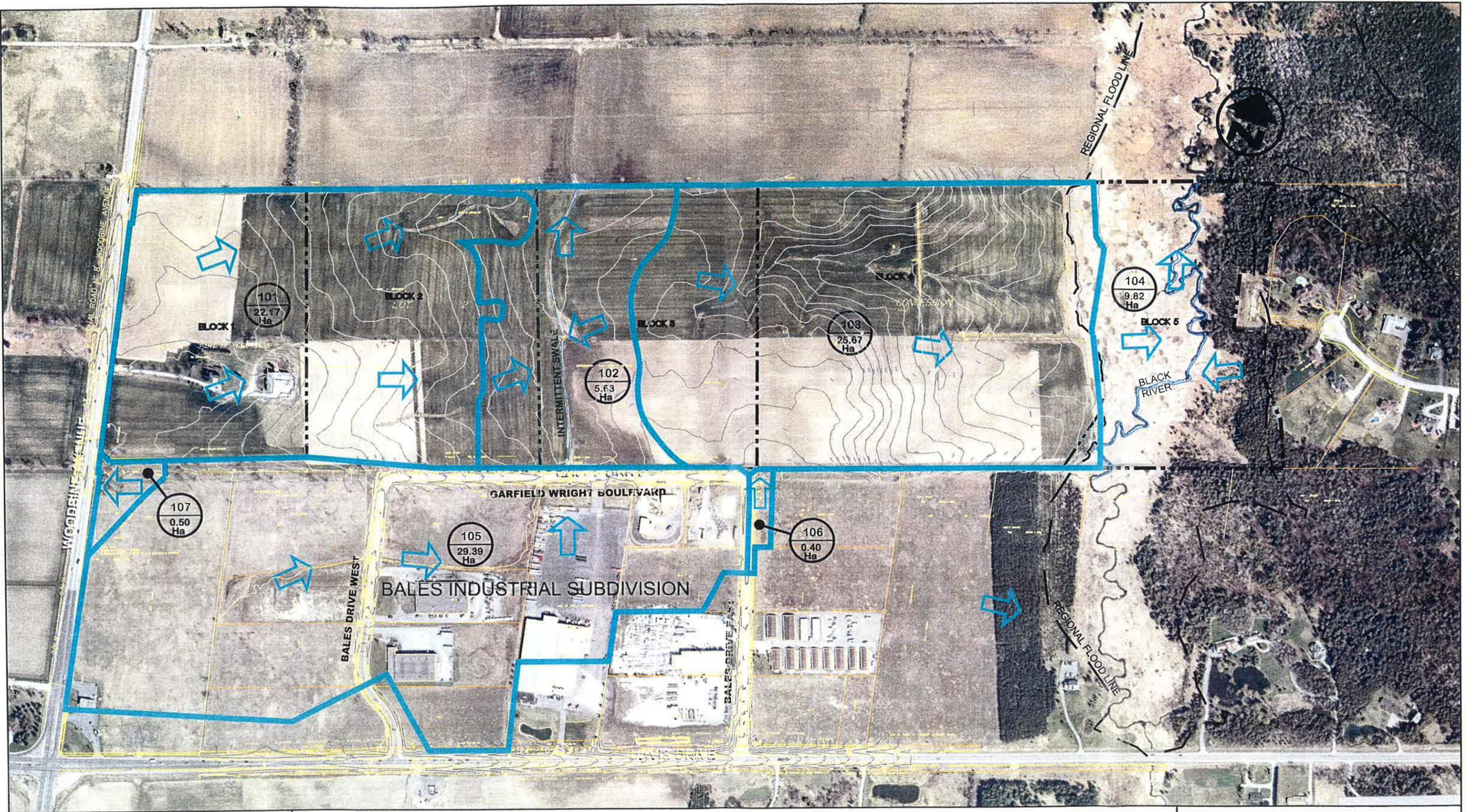
Based on the Black River Subwatershed Study, there is no terrestrial issue and environmentally significant area (ESA) such as biological/hydrogeological ESA, significant wetland, area of natural and scientific interest (ANSI), or provincial archaeological protection area within the subject site and its external area to the south.

In Block 5, there is existing forest along the east side of the Black River as well as open marsh meadow over the balance. Under the interim and ultimate development plans for the York Region Industrial Subdivision, Block 5 is designated as the open space (green buffer area).

## 2.5 Interim and Ultimate Development Plans

Although the entire subject site is designated as the York Region Industrial Subdivision, there is preliminary draft plan of subdivision and two site plans that are ready for immediate development. The staff at the Region and the Town was contacted to clarify the immediate and potential future development plans for the subdivision and the external industrial development areas to the south. Based on the background information available at this time, the immediate (interim) and future (ultimate) developments that will be serviced by SWMF1 and SWMF2 (located at the northeast corner of Block 2 and at the southeast corner of Block 4 respectively) can be summarized in **Table 1**. The detailed interim and ultimate development plans are illustrated in **Figures 5 and 6** respectively.

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**YORK REGION INDUSTRIAL SUBDIVISION**  
 Town of East Gwillimbury, Regional Municipality of York

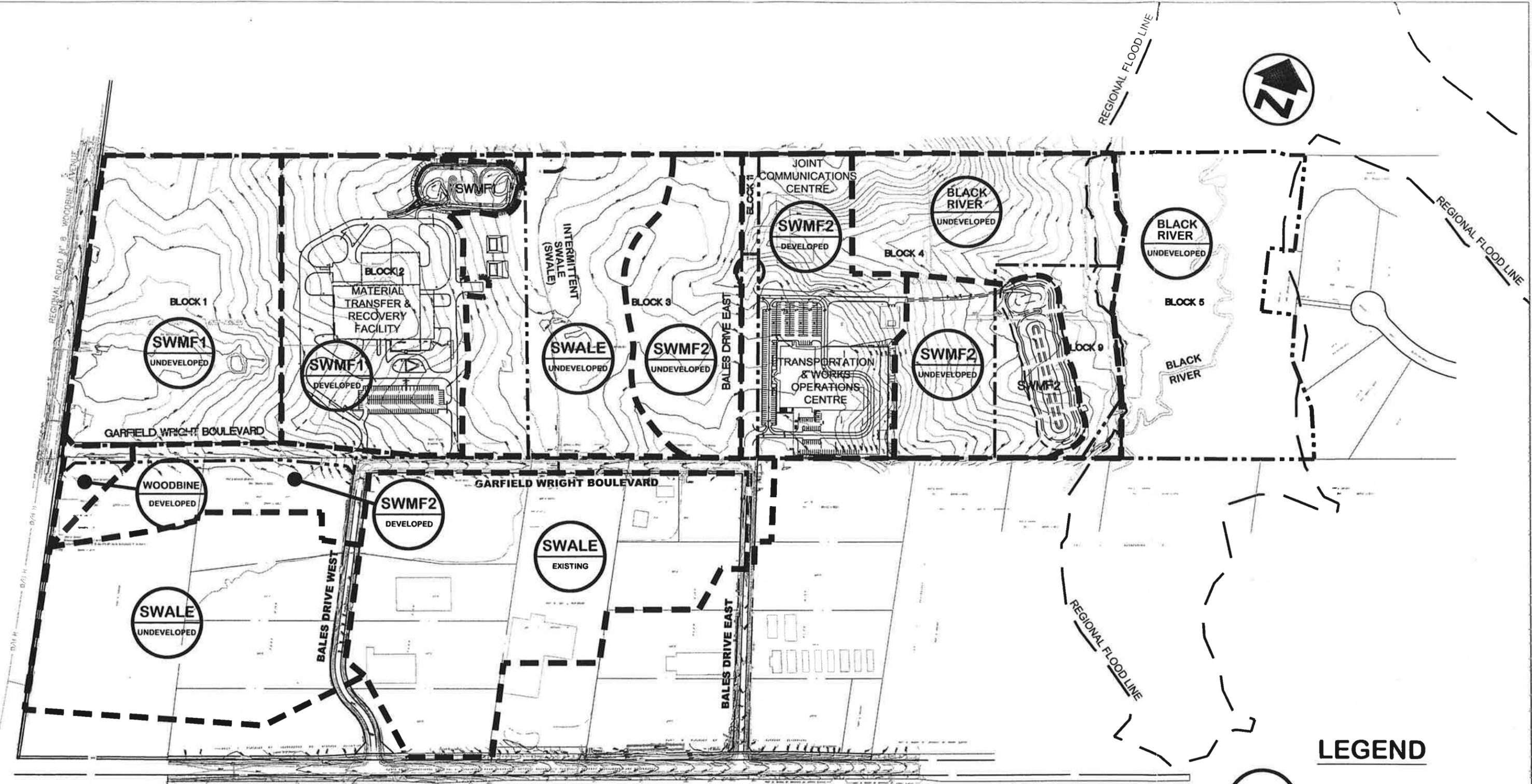


**FIGURE 4 EXISTING DRAINAGE CONDITIONS**

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**LEGEND**

-  DENOTES DRAINAGE DESTINATION
-  DENOTES DESIGN DEVELOPMENT STATUS



**YORK REGION INDUSTRIAL SUBDIVISION**  
Town of East Gwillimbury, Regional Municipality of York

**FIGURE 5 INTERIM DEVELOPMENT PLAN**

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**Table 1. Interim and Ultimate Developments Serviced by SWMF1 and SWMF2**

<b>SWM Facility</b>	<b>Developments under Interim Conditions</b>	<b>Developments under Ultimate Conditions</b>
<b>SWMF1</b>	<ul style="list-style-type: none"> <li>York Region Material Recovery and Transfer Facility in Block 2.</li> </ul>	<ul style="list-style-type: none"> <li>All developments under the interim conditions.</li> <li>Full industrial development in Block 1.</li> </ul>
<b>SWMF2</b>	<ul style="list-style-type: none"> <li>Transportation and Works Operations Centre in Block 4.</li> <li>Joint Communications Centre in Block 4.</li> <li>Zenon Sewage Treatment Plant in Block 4.</li> <li>North extension of Bales Drive East.</li> <li>Urbanization of existing Garfield Wright Blvd.</li> <li>West extension of Garfield Wright Blvd from Bales Drive West to Woodbine Avenue, and the small at the south side of the extended portion of Garfield Wright Blvd.</li> </ul>	<ul style="list-style-type: none"> <li>All developments under the interim conditions.</li> <li>Full industrial development within the east part of Block 3 to maximize developable lands.</li> <li>Full industrial development for the balance of Block 4, excluding the easement Block 9 designated and reserved to accommodate the ultimate SWMF2.</li> </ul>

**Notes:**

- SWM: stormwater management.
- SWMF: stormwater management facility.
- SWMF1: the first stormwater management facility (located at the northeast corner of Block 2) to accommodate the ultimate industrial development within Blocks 1 and 2. The design of the ultimate SWMF1 had been completed by Marshall Macklin Monaghan Limited in Oct. 2003 on behalf of Miller Waste Systems, and approved by the regulatory agencies.
- SWMF2: the second stormwater management facility (located at the southeast corner of Block 4) to accommodate the industrial development within Block 4, the major portion of Block 3 and the areas associated with the extension and urbanization of Garfield Wright Blvd to Woodbine Ave. The detailed engineering design of the interim SWMF2 is detailed in this Report and the attached engineering design drawings, in addition to the preliminary design of the ultimate SWMF2 with a designated stormwater management easement block (Block 9) to accommodate the ultimate SWMF2.
- The detailed interim and ultimate development plans are illustrated in **Figures 5 and 6**.

## 3.0 DESIGN OF STORMWATER MANAGEMENT FACILITIES

### 3.1 Stormwater Management Design Criteria

The following criteria were identified for the design of stormwater management facilities:

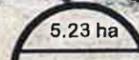
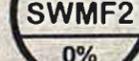
- Respect the regulatory floodplain of the Black River.
- Provide *Enhanced* (Level 1) water quality protection.
- Detain runoff from the 25-mm design storm over a period of 24 hours for erosion control.
- Control the post-development peak flows to or below the pre-development equivalents for the 1:2 to 1:100 year design storms.
- The minor drainage system designed to convey runoff up to the 1:5 year design storm.
- The major drainage system designed to convey runoff up to the 1:100 year design storm, and the water quantity control storage required for the pond checked by the 24-hour SCS storms.
- Minimize erosion potential at stormwater drainage outlets where flow becomes concentrated.
- Provide temporary erosion and sedimentation control during construction.

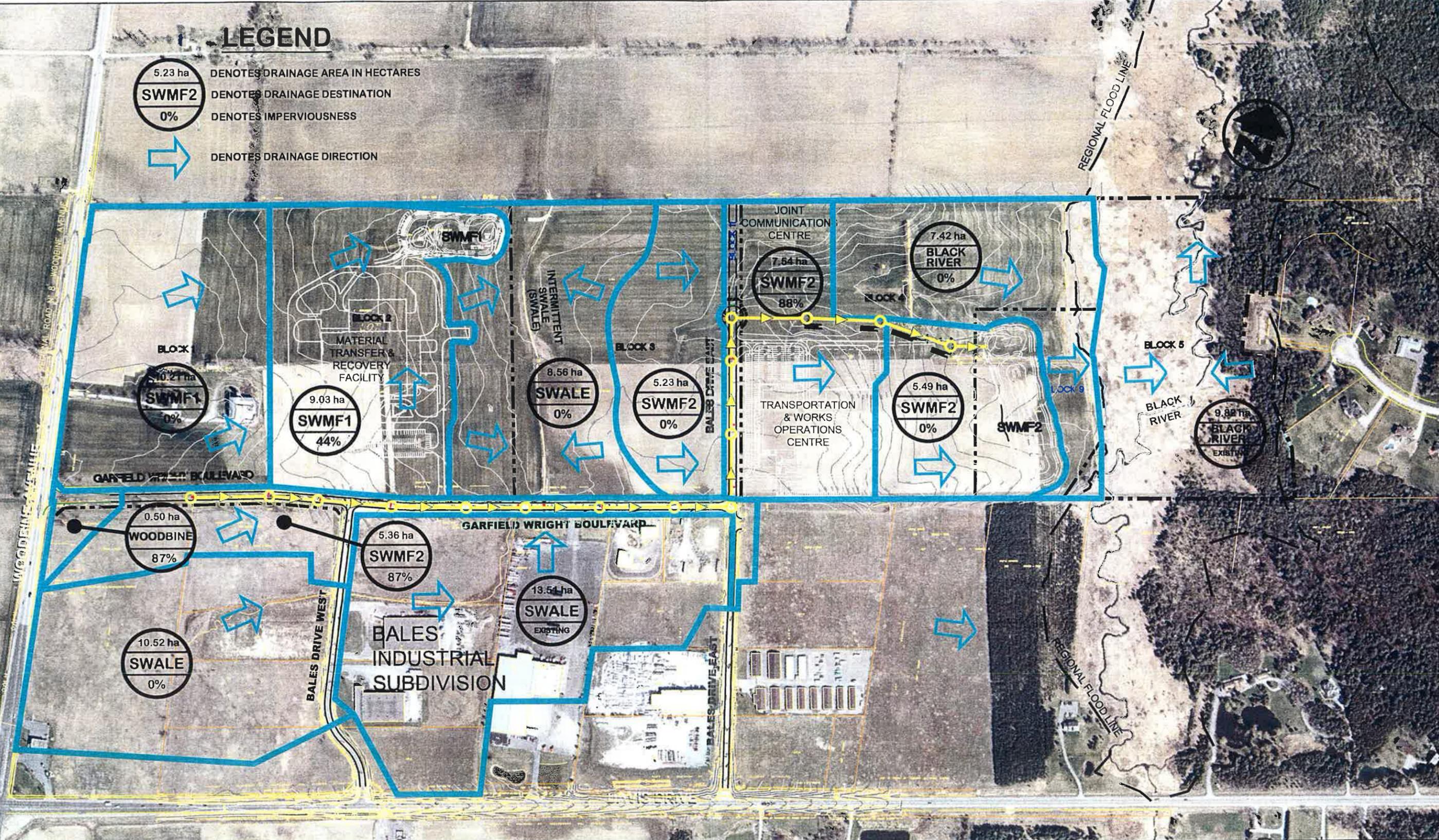
### 3.2 Interim and Ultimate Drainage Conditions

According to the interim and ultimate development plans (see **Figures 5 and 6**), the interim and ultimate drainage patterns in relationship with the facilities of SWMF1 and SWMF2 are proposed and presented in **Figures 7 and 8** respectively. As concluded in the Report entitled "Stormwater Management Plan for the York Region Waste Transfer Station" completed by Marshall Macklin Monaghan Ltd. in Oct. 2003 and approved by the regulatory agencies, the ultimate SWMF1 has been designed to accommodate the ultimate industrial development within Block 1 and Block 2.

SWMF2 is to accommodate the interim and ultimate industrial developments within Block 4, the major portion of Block 3 and the area associated with the extension and urbanization of Garfield Wright Blvd. Under this arrangement, the current drainage conditions within the existing Bales Industrial Subdivision and the majority of the area to its west will remain draining toward the intermittent swale as the source of water supply to the existing fish habitat along the intermittent swale. Only the minimum area associated with the extension and urbanization of Garfield Wright Blvd will be drained into SWMF2 for erosion, water quality and water quantity control. Because the extension of Garfield Wright Blvd to Woodbine Ave. and the small area to its south belong to the new developments, the peak flows up to the 1:100 year return period from these areas are to be conveyed via the storm sewer system to SWMF2 for water quantity control. It should be noted that, as required by the LSRCA, new development within the existing Bales Industrial Subdivision and the area to its west must provide *enhanced* water quality, water quantity and erosion controls before releasing water into the intermittent swale to prevent the flooding, protect the existing fish habitat, and the ecosystem along the swale.

# LEGEND

-  DENOTES DRAINAGE AREA IN HECTARES
-  DENOTES DRAINAGE DESTINATION
-  DENOTES IMPERVIOUSNESS
-  DENOTES DRAINAGE DIRECTION



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## YORK REGION INDUSTRIAL SUBDIVISION Town of East Gwillimbury, Regional Municipality of York

FIGURE 7 INTERIM POST-DEVELOPMENT DRAINAGE CONDITIONS

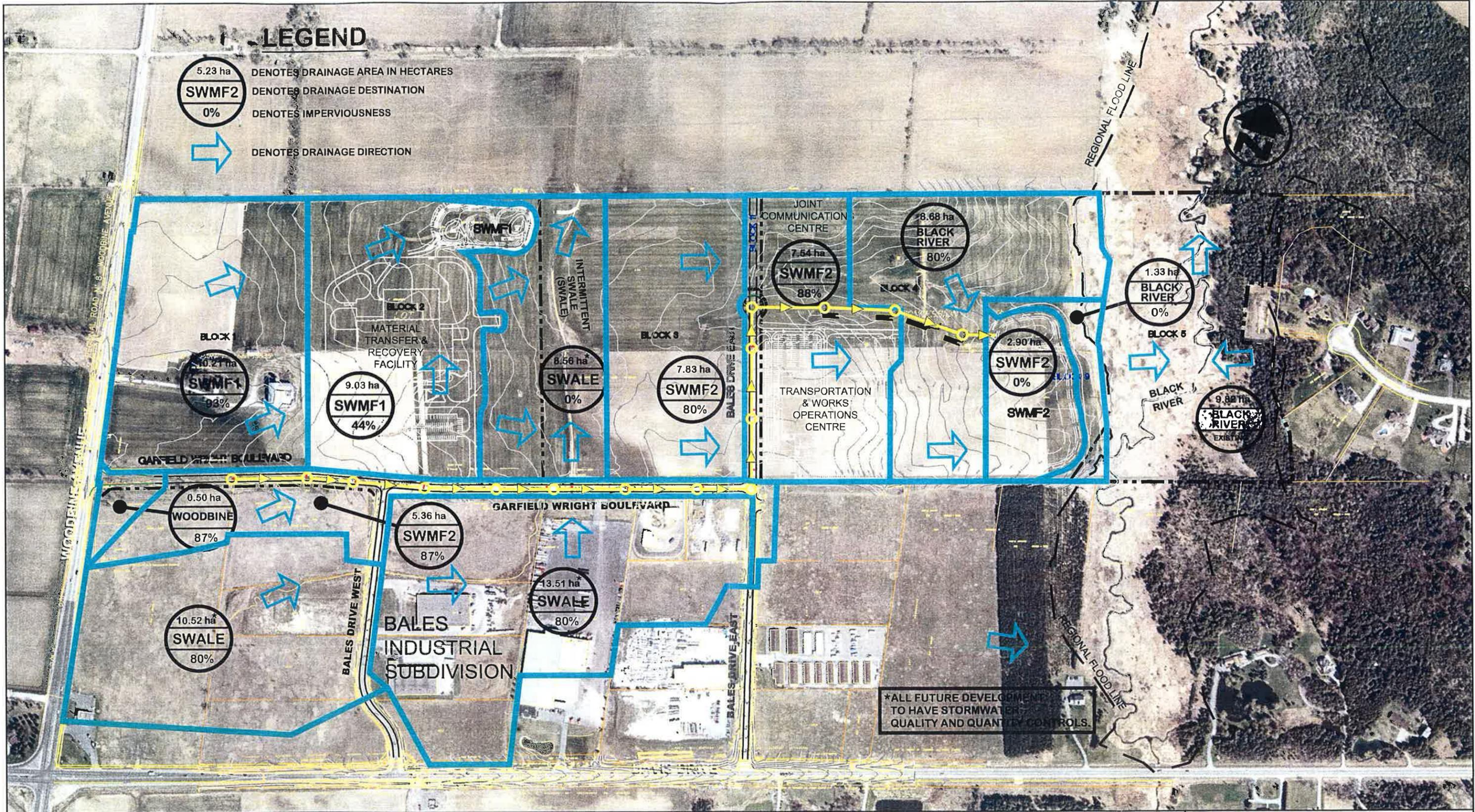
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# LEGEND

- DENOTES DRAINAGE AREA IN HECTARES
- DENOTES DRAINAGE DESTINATION
- DENOTES IMPERVIOUSNESS
- DENOTES DRAINAGE DIRECTION



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## YORK REGION INDUSTRIAL SUBDIVISION

Town of East Gwillimbury, Regional Municipality of York

FIGURE 8 ULTIMATE POST-DEVELOPMENT DRAINAGE CONDITIONS

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### 3.3 Engineering Design of the Interim SWMF2

The interim SWMF2 is designed to accommodate the interim development (including the partial development in Block 4, and the areas associated with the urbanization and extension of Garfield Wright Blvd) as illustrated in **Figure 5**. The development status and drainage areas controlled by the interim SWMF2 are shown in **Figure 7**, and the major design parameters for the interim SWMF2 are detailed in **Table 2**.

**Table 2. Major Design Parameters for the Interim SWMF2**

SWM Facility	Development / Drainage Contributing Area	Development Status	Area (ha)	Imperviousness (%)	CN* (-)
Interim SWMF2	East portion of Block 3 located at the west side of the extension part of Bales Drive East	Undeveloped	5.23	0%	62
	Transportation and Works Operations Centre; North extension of Bales Drive East; Joint Communications Centre; and Zenon Sewage Treatment Plant in Block 4	Developed	7.54	88%	35
	The balance of Block 4 draining towards the interim SWMF2, located at the east side of Transportation and Works Operations Centre and including the interim SWMF2 area	Undeveloped except for SWMF2	5.49	0%	35
	The areas associated with the extension and urbanization of Garfield Wright Blvd to Woodbine Ave.	Developed	5.36	87%	62

**Notes:**

- SWM: stormwater management.
- Interim SWMF2: the interim stormwater management facility located at the southeast corner of Block 4 to provide the stormwater management control for the interim development plan (as described in **Table 1** and illustrated in **Figures 5 and 7**) for the industrial development in Block 4, the major portion of Block 3 and the areas associated with the extension and urbanization of Garfield Wright Blvd.
- CN\*: modified runoff curve number for the pervious area and its calculation details are presented in **Appendix D**.
- Development status: whether or not the area is considered as the developed area for the design of the interim SWMF2.

The interim SWMF2 is located at the southeast corner of Block 4 and it is a deep extended wet pond with a sediment forebay in front. Both wet pond and forebay are situated entirely outside of the regulatory floodplain of the Black River. As illustrated in **Figure 7**, the total drainage area to the interim SWMF2 is approximately 23.6 ha, including the 12.9 ha development area with the average imperviousness of 87.5% and the 10.7 ha undeveloped area (including SWMF2 itself). SWMF2 treats and attenuates the post-development stormwater runoff from the development site and discharges the treated water into the Black River directly at the controlled rates to satisfy the desired design criteria.

**Sediment Forebay:** The sediment forebay is designed to facilitate operation and maintenance of the wet pond and improve pollutant removal by trapping large sediments from entering the pond. Considering the 12.9 ha development area with an average imperviousness of 87.5%, the annual sediment loading to the forebay is about 51.2 m<sup>3</sup>/year (3.97 m<sup>3</sup>/ha/year), according to the current MOE Stormwater Management Planning and Design Manual published in March 2003.

The procedure recommended in the MOE Stormwater Management Planning and Design Manual was followed to design the sediment forebay as detailed in **Appendix D**. According to the current

design, the 2.0 m deep sediment forebay provides a total storage volume over 1,631 m<sup>3</sup> with an average cleanout frequency exceeding 10 years.

**Storage Requirement:** Based on the desired design criteria, the storage required for the interim SWMF2 should satisfy the following criteria:

- Storage required for *water quality control*: in accordance with the current MOE standards, the storage required to provide *Enhanced* water quality protection under the interim development conditions should be the maximum of the following storage volume requirements:
  - 1) for the 12.9 ha development area with 87.5% imperviousness:
    - permanent pool required: 2,763 m<sup>3</sup> (214.2 m<sup>3</sup>/ha)
    - extended detention required: 516 m<sup>3</sup> (40.0 m<sup>3</sup>/ha)
    - total storage volume required: 3,279 m<sup>3</sup> (254.2 m<sup>3</sup>/ha)
  - 2) for the 23.62 ha drainage area to SWMF2 with 47.8% imperviousness:
    - permanent pool required: 3,118 m<sup>3</sup> (132.0 m<sup>3</sup>/ha)
    - extended detention required: 945 m<sup>3</sup> (40.0 m<sup>3</sup>/ha)
    - total storage volume required: 4,063 m<sup>3</sup> (172.0 m<sup>3</sup>/ha)
- Storage required for *erosion control*: the post-development stormwater runoff produced from the development site under the 25-mm design storm must be detained over 24 hours. Based on the hydrologic analysis, the storage requirement for erosion control is about 3,120 m<sup>3</sup>.
- Storage required for *flood control*: the post-development peak flows released from SWMF2 is to be controlled to or below the pre-development equivalents for the 1:2 to 1:100 year design storms. Based on the hydrologic analysis, the storage requirement for flood control up to the 1:100 year design storm is about 10,110 m<sup>3</sup>.

Therefore, the total storage required for the interim SWMF2 is approximately 13,228 m<sup>3</sup>, including the permanent pool of 3,118 m<sup>3</sup> and the extended detention of 10,110 m<sup>3</sup> (excluding the storage created by the free board). Based on the current design, the interim SWMF2 provides a total storage over 24,884 m<sup>3</sup>, including the permanent pool of 10,066 m<sup>3</sup> and the active storage of 14,818 m<sup>3</sup> (including the storage created by the free board). The average depth of the wet pond is 4.60 m, including the 3.0 m permanent pool as requested by the LSRCA with bottom draws for the coldwater fisheries and the 1.6 m deep active storage.

To maintain the permanent pool within the wet pond and the sediment forebay, a 1.0 m thick clay liner (that must be uniformly compacted to achieve at least 95% of its maximum Standard Proctor dry density) is suggested to be placed on their bottoms and slopes slightly above the permanent pool level and surrounded by subdrain draining toward the Black River. As requested by the LSRCA, the 300 mm thick topsoil will be placed on the slopes of the wet pond and the forebay above the permanent pool for landscaping.

**Outlet Structure:** The outlet structure of the interim SWMF2 is designed, through a trial-and-error approach, to satisfy the design criteria required to provide *Enhanced* water quality, erosion and flood control. According to the design, the outlet structure is located at the south end of the wet pond to increase the length of the flow path and consists of a 180 mm diameter orifice (at the outlet end of the reversed pipe to draw cold water from the bottom of the wet pond) with the invert elevation of 266.00 m; a 340 mm diameter orifice (at the inlet end of a 675 mm diameter concrete outlet pipe connected to the DICB structure) with the invert elevation of 265.60 m used only when the water level in the wet pond is above 266.60 m; and a 8.0 m wide overflow spillway crested at the elevation of 267.20 m for emergency spill. The detailed stage-discharge-storage relationship of the interim SWMF2 is presented in **Table 3**. The schematics illustrating the overall grading and outlet details of the interim SWMF2 are included **Appendix D**.

**Drawdown Time:** Based on the design configuration of the interim SWMF2, the drawdown time of SWMF2 under the 25-mm design storm is about 34 hours, calculated using the MOE approach as detailed in **Appendix D**, and it satisfies the minimum drawdown time of 24 hrs as required.

**Operation and Maintenance:** Proper operation and regular maintenance are essential to ensure optimal performance of the facility as designed. As SWMF2 consists of the sediment forebay and wet pond, operation and maintenance activities mainly include monitoring and inspections, weed control, grass cutting, upland and aquatic vegetation replanting, cleaning and adjustment of the outlet, removal of accumulated sediments, shoreline and flood fringe vegetation replanting, and trash removal for the pond, forebay and all inlet and outlet structures, excluding the operation and maintenance activities for temporary erosion and sediment controls for construction as described in **Appendix D**.

The inspections and monitoring of the facility determine required maintenance activities, and they include many aspects such as the hydraulic performance of the facility, slope stability, conditions of vegetation in and around the facility, evidence of spill and oil/grease contamination, measured sediment depth and frequency of sediment and trash build-up among others. During the first two years of operation, it is recommended inspections to be made after every significant storm to ensure proper functioning of the facility (on average about four inspections per year). After this initial period, when the designed operation and performance of SWMF2 have been achieved or confirmed, annual inspections may suffice.

Generally speaking, it is recommended to limit grass cutting and weed control (governed by local by-laws) around SWMF2 since grass growth tends to enhance water quality treatment, provide ideal wildlife habitat among other benefits for the wet pond. Grass around the wet pond and the forebay should not be cut to the edge of the permanent pool. As a safety precaution, grass cutting should be done parallel to the shoreline with grass clipping being ejected upland to reduce the potential for organic loadings to the wet pond.

Table 3. Stage-Storage-Discharge Relationship of the Interim SWMF2

Elevation (m)	Depth (m)	Permanent Pool Storage (m <sup>3</sup> )	Active Storage (m <sup>3</sup> )	Total Storage (m <sup>3</sup> )	Orifice A <sup>(2)</sup> Outflow (m <sup>3</sup> /s)	Orifice B <sup>(2)</sup> Outflow (m <sup>3</sup> /s)	Emergency Spillway (m <sup>3</sup> /s)	Total Outflow (m <sup>3</sup> /s)
263.00	0.00	0	0	0	0.000	0.000	0.000	0.000
263.10	0.10	174	0	174	0.000	0.000	0.000	0.000
263.20	0.20	357	0	357	0.000	0.000	0.000	0.000
263.30	0.30	549	0	549	0.000	0.000	0.000	0.000
263.40	0.40	751	0	751	0.000	0.000	0.000	0.000
263.50	0.50	963	0	963	0.000	0.000	0.000	0.000
263.60	0.60	1,185	0	1,185	0.000	0.000	0.000	0.000
263.70	0.70	1,418	0	1,418	0.000	0.000	0.000	0.000
263.80	0.80	1,660	0	1,660	0.000	0.000	0.000	0.000
263.90	0.90	1,914	0	1,914	0.000	0.000	0.000	0.000
264.00	1.00	2,179	0	2,179	0.000	0.000	0.000	0.000
264.10	1.10	2,454	0	2,454	0.000	0.000	0.000	0.000
264.20	1.20	2,741	0	2,741	0.000	0.000	0.000	0.000
264.30	1.30	3,039	0	3,039	0.000	0.000	0.000	0.000
264.40	1.40	3,350	0	3,350	0.000	0.000	0.000	0.000
264.50	1.50	3,672	0	3,672	0.000	0.000	0.000	0.000
264.60	1.60	4,006	0	4,006	0.000	0.000	0.000	0.000
264.70	1.70	4,352	0	4,352	0.000	0.000	0.000	0.000
264.80	1.80	4,711	0	4,711	0.000	0.000	0.000	0.000
264.90	1.90	5,083	0	5,083	0.000	0.000	0.000	0.000
265.00	2.00	5,467	0	5,467	0.000	0.000	0.000	0.000
265.10	2.10	5,865	0	5,865	0.000	0.000	0.000	0.000
265.20	2.20	6,276	0	6,276	0.000	0.000	0.000	0.000
265.30	2.30	6,700	0	6,700	0.000	0.000	0.000	0.000
265.40	2.40	7,138	0	7,138	0.000	0.000	0.000	0.000
265.50	2.50	7,590	0	7,590	0.000	0.000	0.000	0.000
265.60	2.60	8,056	0	8,056	0.000	0.000	0.000	0.000
265.70	2.70	8,537	0	8,537	0.000	0.000	0.000	0.000
265.80	2.80	9,031	0	9,031	0.000	0.000	0.000	0.000
265.90	2.90	9,541	0	9,541	0.000	0.000	0.000	0.000
266.00	3.00	10,066	0	10,066	0.000	0.000	0.000	0.000
266.10	3.10	10,066	702	10,767	0.007	0.000	0.000	0.007
266.20	3.20	10,066	1,427	11,492	0.022	0.000	0.000	0.022
266.30	3.30	10,066	2,176	12,241	0.031	0.000	0.000	0.031
266.40	3.40	10,066	2,948	13,013	0.038	0.000	0.000	0.038
266.50	3.50	10,066	3,744	13,810	0.043	0.000	0.000	0.043
266.60	3.60	10,066	4,649	14,715	0.048	0.000	0.000	0.048
266.70	3.70	10,066	5,574	15,640	-	0.233	0.000	0.233
266.80	3.80	10,066	6,519	16,584	-	0.245	0.000	0.245
266.90	3.90	10,066	7,484	17,549	-	0.257	0.000	0.257
267.00	4.00	10,066	8,469	18,535	-	0.268	0.000	0.268
267.10	4.10	10,066	9,475	19,540	-	0.278	0.000	0.278
267.20	4.20	10,066	10,501	20,567	-	0.289	0.000	0.289
267.30	4.30	10,066	11,548	21,614	-	0.298	0.430	0.729
267.40	4.40	10,066	12,617	22,682	-	0.308	1.216	1.524
267.50	4.50	10,066	13,707	23,772	-	0.317	2.235	2.552
267.60	4.60	10,066	14,818	24,884	-	0.326	3.441	3.767

**Design Specifications of the Interim SWMF2:**

- (1) *Extended Wet Pond:* top of berm elevation of 267.60 m, bottom elevation of 263.00 m, permanent pool level of 266.00 m, average wet pond depth of 4.60 m (including 3.00 m for the permanent pool as requested by the LSRCA and 1.60 m for the active storage), and the average side slope of 5:1 (Horizontal:Vertical).
- (2) *Outlet Structure:* one 180 mm diameter orifice (Orifice A) at the invert elevation of 266.00 m at the outlet end of the reversed pipe, one 340 mm diameter orifice (Orifice B) at the invert elevation of 265.60 m at the inlet end of the outflow pipe from the DICB structure, and one 8.0 m wide emergency overflow spillway crested at the elevation of 267.20 m to discharge the treated water directly into the Black River.
- (3) Orifice flow equation:  $Q = C_d A \sqrt{2gH}$  where Q is the discharge (m<sup>3</sup>/s); C<sub>d</sub> the orifice discharge coefficient (C<sub>d</sub> = 0.6); A the orifice area (m<sup>2</sup>); g the gravitational acceleration constant (9.81 m/s<sup>2</sup>); and H the effective water head above the orifice (m).
- (4) Weir flow equation:  $Q = CBH^{1.5}$  where Q is the discharge (m<sup>3</sup>/s); C the weir flow coefficient (C = 1.7); B the weir width (m); and H the effective head of water above the weir crest (m).

Trash removal is an integrated part of regular maintenance activities, and it is recommended that a "spring cleanup" is at least required to remove trash from the facility, and is then performed as required based on observations during regular inspections.

One of the most important maintenance requirements for the effective performance of SWMF2 is the removal of accumulated sediment, particularly in the forebay. Because the sediment forebay is in front of the wet pond to trap large sediments from entering the pond, the minimum sediment removal is required for the wet pond, except for the portion connected to the outlet structure as the sediment accumulation at the inlet end of the reversed pipe can block the flow from entering the pipe and the flow control orifice. Based on the current design, the sediment forebay has to be cleaned at least once in every 10 years on average, though the sediment removal for the forebay may be required shortly after the completion of the construction activities.

The typical excavation equipment such as the backhoes and hydraulic dredging may be used to remove sediment from the forebay and the wet pond. Because SWMF2 is designed to collect the stormwater runoff from the industrial subdivisions, it is recommended all sediment removed from the wet pond and the sediment forebay to be tested, confirm whether or not it is classified as the hazardous waste and determine alternative disposal options. The current MOE sediment disposal requirements should be consulted for information pertaining to exact parameters and acceptable levels for different disposal options.

### 3.4 Engineering Design of the Ultimate SWMF2

The ultimate SWMF2 is designed to accommodate the ultimate development plan (see **Figure 6**) due to the full development in Blocks 3 and 4, and in the areas associated with the urbanization and extension of Garfield Wright Blvd. The development status and drainage areas controlled by the ultimate SWMF2 are shown in **Figure 8**, and the major design parameters for the ultimate SWMF2 are detailed in **Table 4**.

The ultimate SWMF2 is a deep extended wet pond with a sediment forebay both of which are situated outside of the regulatory floodplain of the Black River. As shown in **Figure 8**, the total drainage area to the ultimate SWMF2 is about 32.3 ha, including the 29.4 ha development area with the average imperviousness of 83.3%. SWMF2 treats and attenuates the stormwater runoff from the development site and discharges the treated water back into the Black River directly at the controlled rates to satisfy the desired design criteria.

**Sediment Forebay:** Considering the 29.4 ha development area with an average imperviousness of 83.3%, the annual sediment loading to the forebay is about 108.5 m<sup>3</sup>/year (3.69 m<sup>3</sup>/ha/year) according to the current MOE Stormwater Management Planning and Design Manual.

The procedure recommended in the MOE Stormwater Management Planning and Design Manual was followed to design the ultimate sediment forebay as given in **Appendix D**. According to the current design, the 2.0 m deep sediment forebay provides a total storage volume over 3,187 m<sup>3</sup> with an average cleanout frequency exceeding 11 years.

**Table 4. Major Design Parameters for the Ultimate SWMF2**

SWM Facility	Development / Drainage Contributing Area	Development Status	Area (ha)	Imperviousness (%)	CN* (-)
Ultimate SWMF2	Major portion of Block 3 at the west side of the extension part of Bales Drive East	Developed	7.83	80%	62
	Transportation and Works Operations Centre; North extension of Bales Drive East; Joint Communications Centre; and Zenon Sewage Treatment Plant in Block 4	Developed	7.54	88%	35
	The balance of Block 4 draining towards the ultimate SWMF2, located at the east half of Block 4 and excluding the ultimate SWMF2 site	Developed	8.68	80%	35
	The site to accommodate the ultimate SWMF2 in Block 4	Undeveloped except for SWMF2	2.90	0%	35
	The areas associated with the extension and urbanization of Garfield Wright Blvd to Woodbine Ave, including the small area to its south	Developed	5.36	87%	62

**Notes:**

- SWM: stormwater management.
- Ultimate SWMF2: the ultimate stormwater management facility located at the southeast corner of Block 4 to provide the stormwater management control for the ultimate development plan (as described in **Table 1** and illustrated in **Figures 6 and 8**) for the industrial development in Block 4, the major portion of Block 3 and the areas associated with the extension and urbanization of Garfield Wright Blvd.
- CN\*: modified runoff curve number for the pervious area and its calculation details are presented in **Appendix D**.
- Development status: whether or not the area is considered as the developed area for the design of the ultimate SWMF2.

**Storage Requirement:** Based on the desired design criteria and other design assumptions, the storage requirement for the ultimate SWMF2 should satisfy the following criteria:

- Storage required for *water quality control*: in accordance with the current MOE criteria, the storage requirement to provide *Enhanced* (Level 1) water quality protection for the 29.4 ha development area with 83.3% imperviousness is 7,270 m<sup>3</sup> (247.2 m<sup>3</sup>/ha), including the permanent pool storage of 6,094 m<sup>3</sup> (207.2 m<sup>3</sup>/ha) and the extended detention storage of 1,176 m<sup>3</sup> (40.0 m<sup>3</sup>/ha).
- Storage required for *erosion control*: the post-development stormwater runoff produced from the development site under the 25-mm design storm must be detained over 24 hours. Based on the hydrologic analysis, the storage requirement for erosion control is about 6,388 m<sup>3</sup>.
- Storage required for *flood control*: the post-development peak flows released from SWMF2 is to be controlled to or below the pre-development equivalents for the 1:2 to 1:100 year design storms. Based on the hydrologic analysis, the storage requirement for flood control up to the 1:100 year design storm is about 20,170 m<sup>3</sup>.

Therefore, the total storage required for the ultimate SWMF2 is about 26,264 m<sup>3</sup>, including the permanent pool of 6,094 m<sup>3</sup> and the extended detention of 20,170 m<sup>3</sup> (excluding the storage created by the free board). Based on the current design, the ultimate SWMF2 provides a total

storage over 56,629 m<sup>3</sup>, including the permanent pool of 26,956 m<sup>3</sup> and the active storage of 29,674 m<sup>3</sup> (including the storage created by the free board). The average depth of the wet pond is 4.60 m, including the 3.0 m permanent pool as requested by the LSRCA with bottom draws for the coldwater fisheries and the 1.6 m deep active storage.

**Outlet Structure:** The outlet structure of the ultimate SWMF2 is designed to satisfy the design criteria required to provide *Enhanced* water quality, erosion and flood control under the ultimate development condition. According to the current design, the proposed outlet structure consists of a 260 mm diameter orifice (at the outlet end of the reversed pipe to draw cold water from the bottom of the wet pond) with the invert elevation of 266.00 m; a 390 mm diameter orifice (at the inlet end of a 675 mm diameter concrete outlet pipe connected to the DICB structure) with the invert elevation of 265.60 m used only when the water level in the wet pond is above 266.60 m; and a 13.0 m wide overflow spillway crested at the elevation of 267.20 m for emergency spill. The detailed stage-discharge-storage relationship of the ultimate SWMF2 is presented in **Table 5**.

The schematics illustrating grading and outlet details of the ultimate SWMF2 are presented in **Appendix D**, and the outlet structure of the ultimate SWMF2 will be constructed through minor modifications of the interim outlet structure as described below:

- at the outlet end of the reversed concrete pipe, install a larger 260 mm diameter orifice at the same location of the 180 mm diameter interim orifice with the same invert elevation of 266.0 m at the permanent pool level;
- at the inlet end of a 675 mm diameter concrete outlet pipe connected to the DICB structure, install a larger 390 mm diameter orifice at the same location of the 340 mm diameter interim orifice with the same invert elevation of 265.6 m; and
- widen the emergency spillway from the interim length of 8.0 m to the ultimate length of 13.0 m at the same weir invert elevation of 267.20 m. As requested by the LSRCA, a 13.0 m long concrete weir should be installed in the berm under the ultimate condition.

It should be noted that the site plans for the future industrial development areas located within Blocks 3 and 4 are not available at this time. When the blocks are developed, they will require site plan approval and the preparation of a storm drainage report to provide a detailed engineering design of the major and minor storm drainage systems and retrofit the interim SWMF2, in accordance with the drainage design criteria outlined in this report, to safely convey runoff into SWMF2 and provide the desired Level 1 (*enhanced*) water quality, erosion and water quantity control for the development.

**Drawdown Time:** According to the design configuration of the ultimate SWMF2 and its outlet, the drawdown time under the 25-mm storm is about 32 hours, calculated using the MOE approach detailed in **Appendix D**, and it satisfies the minimum drawdown time of 24 hrs required.

Table 5. Stage-Storage-Discharge Relationship of the Ultimate SWMF2

Elevation (m)	Depth (m)	Permanent Pool Storage (m <sup>3</sup> )	Active Storage (m <sup>3</sup> )	Total Storage (m <sup>3</sup> )	Orifice A <sup>(2)</sup> Outflow (m <sup>3</sup> /s)	Orifice B <sup>(2)</sup> Outflow (m <sup>3</sup> /s)	Emergency Spillway (m <sup>3</sup> /s)	Total Outflow (m <sup>3</sup> /s)
263.00	0.00	0	0	0	0.000	0.000	0.000	0.000
263.10	0.10	597	0	597	0.000	0.000	0.000	0.000
263.20	0.20	1,214	0	1,214	0.000	0.000	0.000	0.000
263.30	0.30	1,849	0	1,849	0.000	0.000	0.000	0.000
263.40	0.40	2,504	0	2,504	0.000	0.000	0.000	0.000
263.50	0.50	3,178	0	3,178	0.000	0.000	0.000	0.000
263.60	0.60	3,872	0	3,872	0.000	0.000	0.000	0.000
263.70	0.70	4,586	0	4,586	0.000	0.000	0.000	0.000
263.80	0.80	5,320	0	5,320	0.000	0.000	0.000	0.000
263.90	0.90	6,074	0	6,074	0.000	0.000	0.000	0.000
264.00	1.00	6,849	0	6,849	0.000	0.000	0.000	0.000
264.10	1.10	7,644	0	7,644	0.000	0.000	0.000	0.000
264.20	1.20	8,460	0	8,460	0.000	0.000	0.000	0.000
264.30	1.30	9,298	0	9,298	0.000	0.000	0.000	0.000
264.40	1.40	10,156	0	10,156	0.000	0.000	0.000	0.000
264.50	1.50	11,037	0	11,037	0.000	0.000	0.000	0.000
264.60	1.60	11,938	0	11,938	0.000	0.000	0.000	0.000
264.70	1.70	12,862	0	12,862	0.000	0.000	0.000	0.000
264.80	1.80	13,808	0	13,808	0.000	0.000	0.000	0.000
264.90	1.90	14,776	0	14,776	0.000	0.000	0.000	0.000
265.00	2.00	15,767	0	15,767	0.000	0.000	0.000	0.000
265.10	2.10	16,780	0	16,780	0.000	0.000	0.000	0.000
265.20	2.20	17,817	0	17,817	0.000	0.000	0.000	0.000
265.30	2.30	18,876	0	18,876	0.000	0.000	0.000	0.000
265.40	2.40	19,959	0	19,959	0.000	0.000	0.000	0.000
265.50	2.50	21,065	0	21,065	0.000	0.000	0.000	0.000
265.60	2.60	22,195	0	22,195	0.000	0.000	0.000	0.000
265.70	2.70	23,349	0	23,349	0.000	0.000	0.000	0.000
265.80	2.80	24,527	0	24,527	0.000	0.000	0.000	0.000
265.90	2.90	25,729	0	25,729	0.000	0.000	0.000	0.000
266.00	3.00	26,956	0	26,956	0.000	0.000	0.000	0.000
266.10	3.10	26,956	1,516	28,471	0.014	0.000	0.000	0.014
266.20	3.20	26,956	3,066	30,022	0.037	0.000	0.000	0.037
266.30	3.30	26,956	4,652	31,608	0.058	0.000	0.000	0.058
266.40	3.40	26,956	6,274	33,230	0.073	0.000	0.000	0.073
266.50	3.50	26,956	7,932	34,887	0.086	0.000	0.000	0.086
266.60	3.60	26,956	9,754	36,710	0.097	0.000	0.000	0.097
266.70	3.70	26,956	11,607	38,563	-	0.302	0.000	0.302
266.80	3.80	26,956	13,490	40,446	-	0.318	0.000	0.318
266.90	3.90	26,956	15,404	42,359	-	0.334	0.000	0.334
267.00	4.00	26,956	17,349	44,304	-	0.349	0.000	0.349
267.10	4.10	26,956	19,324	46,280	-	0.363	0.000	0.363
267.20	4.20	26,956	21,331	48,286	-	0.376	0.000	0.376
267.30	4.30	26,956	23,369	50,324	-	0.389	0.699	1.088
267.40	4.40	26,956	25,439	52,394	-	0.402	1.977	2.379
267.50	4.50	26,956	27,540	54,496	-	0.415	3.631	4.046
267.60	4.60	26,956	29,674	56,629	-	0.427	5.591	6.017

**Design Specifications of the Ultimate SWMF2:**

- (1) *Extended Wet Pond:* top of berm elevation of 267.60 m, bottom elevation of 263.00 m, permanent pool level of 266.00 m, average wet pond depth of 4.60 m (including 3.00 m for the permanent pool and 1.60 m for the active storage), and the average side slope of 5:1 (Horizontal:Vertical).
- (2) *Outlet Structure:* one 260 mm diameter orifice (Orifice A) at the invert elevation of 266.00 m at the outlet end of the reversed pipe, one 390 mm diameter orifice (Orifice B) at the invert elevation of 265.60 m at the inlet end of the outflow pipe from the DICB structure, and one 13.0 m wide emergency overflow spillway crested at the elevation of 267.20 m to discharge the treated water directly into the Black River.
- (3) Orifice flow equation:  $Q = C_d A \sqrt{2gH}$  where Q is the discharge (m<sup>3</sup>/s); C<sub>d</sub> the orifice discharge coefficient (C<sub>d</sub> = 0.6); A the orifice area (m<sup>2</sup>); g the gravitational acceleration constant (9.81 m/s<sup>2</sup>); and H the effective water head above the orifice (m).
- (4) Weir flow equation:  $Q = CBH^{1.5}$  where Q is the discharge (m<sup>3</sup>/s); C the weir flow coefficient (C = 1.7); B the weir width (m); and H the effective head of water above the weir crest (m).

SWMF2 will be receiving Zenon Sewage Treatment Plant effluent discharge. The plant will be producing water suitable and used for recycling within the building, with only that not required being discharged. The effluent is of a high quality, normally better than that of receiving streams. The ultimately expected design discharge from the plant is 6,750 l/day, with the interim condition being about half. The ultimate discharge of 6,750 l/day corresponds to 0.0781 l/s (or 0.0000781 m<sup>3</sup>/s) that is well below the level of significance of the design inflow to SWMF2, and therefore implicitly included in the design flows.

It should be noted that several safety signs are suggested around the SWMF2 site and contain the warning, for example: "Warning of Hazardous Conditions: This is a stormwater management facility and contains features which may become potentially hazardous under certain condition. Hazards include pollutants, fluctuating water levels and thin ice within the facility. Please exercise extreme caution within this area. For further information, please contact the Town of East Gwillimbury at (905) 478-4282".

## 4.0 HYDROLOGIC ANALYSIS

### 4.1 Hydrologic Model

Both pre- and post-development hydrologic analyses were conducted by using the OTTHYMO model. The model has been widely used in similar analyses for stormwater management across Ontario and is recognized as a reliable modelling tool to estimate the hydrologic response to both rural and urban watersheds.

The inputs to the OTTHYMO model include meteorological and physiographic data to describe the hydraulic and hydrologic response of the watershed or the stormwater management facilities to the design storms. The major input of the model includes the drainage area, rainfall intensity, soil cover complex curve number, time to peak, an average slope of the catchment and channel, the Manning's roughness coefficient, imperviousness and so on.

### 4.2 Design Storms

The design storms were determined based on the intensity-duration-frequency (IDF) curves using the rainfall data obtained from the Atmosphere Environment Service Station (AES) at Oak Ridges located at the latitude of 43° 58', longitude of 79° 28', and the altitude of 320 m. A summary of the rainfall depths for the complete range of the design storm events is presented in **Table 6**, and the coefficients of the Chicago distribution for the complete range of the design storms are given in **Table 7**. The six-hour Chicago storms were used for the hydrologic analysis. To be conservative, the storage volume requirement of SWMF2 for the water quantity control were checked using the hurricane Hazel and 24-hour SCS Type II distributions under the various return periods.

**Table 6. Rainfall Depths (mm) under Different Design Storm Events**

Storm Duration	1:2 Year Storm	1:5 Year Storm	1:10 Year Storm	1:25 Year Storm	1:50 Year Storm	1:100 Year Storm
5 min	8.7	11.2	12.8	14.8	16.3	17.8
10 min	13.0	17.8	21.0	25.1	28.1	31.1
15 min	16.0	22.6	26.9	32.4	36.4	40.5
30 min	20.3	29.3	35.3	42.9	48.5	54.1
1 hour	23.9	34.6	41.7	50.7	57.4	64.0
2 hours	27.9	38.3	45.2	53.9	60.3	66.7
6 hours	36.1	51.3	61.4	74.1	83.6	92.9
12 hours	42.1	57.2	67.2	79.9	89.3	98.6
24 hours	49.3	67.5	79.6	94.9	106.2	117.5

Source: Atmosphere Environment Service (AES) Oak Ridges Station

Table 7. Main Parameters for Different Design Storm Events

Return Interval (-)	Coefficient A (mm)	Coefficient B (min)	Coefficient C (-)	Peak Intensity (mm/hr)
1:2	686.505	5.262	0.800	106.58
1:5	1172.811	7.969	0.833	138.73
1:10	1516.904	9.188	0.847	160.43
1:25	1960.671	10.512	0.858	186.56
1:50	2282.269	10.910	0.865	208.41
1:100	2630.188	11.602	0.870	228.27

Source: Atmosphere Environment Service (AES) Station at Oak Ridges

### 4.3 Pre-Development Peak Flows

As illustrated in **Figure 4**, the entire study area is divided into several catchments under the pre-development conditions for the hydrologic analysis. Because the post-development runoff from all developments interested drains into the facility SWMF2 located at the southeast corner of Block 4, the pre-development hydrologic analysis focused on Catchment 103 that includes Block 4 and the east portion of Block 3 with a total area of approximately 25.7 ha.

Several parameters such as the weighted modified soil cover complex number, catchment slope, and time to peak were used to simulate the peak flow and the runoff volume under the different design storms. The catchment slope (1.7% for Catchment 103) is calculated using the Equivalent Slope Method suggested in the MTO Drainage Management Manual with the following equation:

$$S_w = 100 \cdot \left[ n / \sum (S_n^{0.5}) \right]^2$$

where:  $S_w$  is the watershed slope (%).

$S_n$  is the slope of an individual reach of the channel (m/m).

$n$  is the number of reaches of approximately equal length (-).

To estimate time to peak (0.24 hours for Catchment 103), the following three-parameter equation is used for catchments with slopes less than 2% and the two-parameter equation for catchments with slopes greater than 2% as suggested in the MTO Drainage Management Manual:

Three-parameter equation:  $t_p = 0.0086 * A^{0.422} * S^{-0.46} * (L/W)^{0.133}$

Two-parameter equation:  $t_p = 0.016 * A^{0.31} * S^{-0.5}$

where:  $t_p$  is the time to peak (hour).

$A$  is the drainage area (ha).

$S$  is the slope (m/m).

$L$  is the catchment length (m).

W is the catchment width (m).

**Table 8** presents the pre-development peak flows for Basin 103 under the 1:2 to 1:100 year Chicago and SCS II design storms. A copy of the complete pre-development OTTHYMO output under the 25 mm, 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year Chicago and SCS II design storms is included in **Appendix B**.

**Table 8. Pre-Development Peak Flows under Different Design Storms**

Items and Specifications	Peak Flows (m <sup>3</sup> /s)					
	1:2 Year Storm	1:5 Year Storm	1:10 Year Storm	1:25 Year Storm	1:50 Year Storm	1:100 Year Storm
Basin 103 for a total area of 25.67 ha						
under the Chicago Storms	0.15	0.30	0.42	0.61	0.76	0.94
under the SCS II Storms	0.18	0.33	0.46	0.64	0.79	0.95
<b>Notes:</b>						
<ul style="list-style-type: none"> <li>• Locations of different catchments under the pre-development conditions are illustrated in <b>Figure 4</b>.</li> <li>• Design Storm Events: six-hour Chicago storms and 24-hour SCS II storms at the AES Oak Ridges Station.</li> <li>• A hard copy of the pre-development OTTHYMO output is included in <b>Appendix B</b>.</li> </ul>						

#### 4.4 Performance of the Interim and Ultimate SWMF2

**Figures 7 and 8** present the interim and ultimate drainage conditions. In accordance with the interim and ultimate development plans and design assumptions described in **Table 2** and **Table 4**, the OTTHYMO model was used to simulate the uncontrolled and controlled post-development peak flows and the performance of SWMF2 under the Hurricane Hazel, Chicago and SCS II rainfall storms. The results of the post-development peak flows and the performance of SWMF2 under the different design storms are summarized in **Table 9** and **Table 10** for the interim and ultimate conditions respectively. A copy of the post-development OTTHYMO output for the 25 mm, 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year Chicago and SCS II storms and the Hurricane Hazel is included in **Appendix C** under both interim and ultimate development conditions.

Table 9. Performance of Interim SWMF2 under Different Design Storms

Technical Parameter Specifications	Rainfall Storm Events						
	1:2 Year	1:5 Year	1:10 Year	1:25 Year	1:50 Year	1:100 Year	Hazel
<b>PERFORMANCE OF INTERIM SWMF2 UNDER 6-HOUR CHICAGO DESIGN STORMS</b>							
Peak inflow to SWMF2 (m <sup>3</sup> /s)	2.74	3.78	4.49	5.39	6.12	6.81	3.07
Peak outflow from SWMF2 (m <sup>3</sup> /s)	0.05	0.18	0.24	0.26	0.27	0.29	2.85
Allowable peak flow* (m <sup>3</sup> /s)	0.11	0.21	0.30	0.43	0.54	0.67	-
Max water level in SWMF2 (m)	266.54	266.67	266.76	266.93	267.05	267.18	267.53
Max storage used in SWMF2 (m <sup>3</sup> )	4,110	5,320	6,180	7,770	8,950	10,290	13,990
Max W/L above the PP (m)	0.54	0.67	0.76	0.93	1.05	1.18	1.53
Max W/depth above the PB (m)	3.54	3.67	3.76	3.93	4.05	4.18	4.53
<b>PERFORMANCE OF INTERIM SWMF2 UNDER 24-HOUR SCS TYPE II STORMS</b>							
Peak inflow to SWMF2 (m <sup>3</sup> /s)	1.11	1.59	1.94	2.37	2.73	3.07	-
Peak outflow from SWMF2 (m <sup>3</sup> /s)	0.06	0.24	0.25	0.27	0.28	0.45	-
Allowable peak flow* (m <sup>3</sup> /s)	0.13	0.24	0.32	0.45	0.56	0.68	-
Max water level in SWMF2 (m)	266.61	266.72	266.84	267.01	267.14	267.24	-
Max storage used in SWMF2 (m <sup>3</sup> )	4,720	5,750	6,920	8,550	9,870	10,930	-
Max W/L above the PP (m)	0.61	0.72	0.84	1.01	1.14	1.24	-
Max W/depth above the PB (m)	3.61	3.72	3.84	4.01	4.14	4.24	-
<b>Notes:</b>							
<ul style="list-style-type: none"> <li>The interim post-development drainage conditions are illustrated in <b>Figure 7</b>.</li> <li>W/L: water level; Max: maximum; PP: permanent pool; W/depth: water depth; PB: the bottom of the wet pond.</li> <li>*: allowable peak flows are estimated by multiplying the peak flows presented in <b>Table 8</b> by an area reduction factor of <math>18.25/25.67 = 0.71</math> because there is a small area (7.42 ha) out of 25.67 ha draining directly into the Black River rather than into SWMF2 under the interim development condition.</li> <li>A hard copy of the interim OTTHYMO output is included in <b>Appendix C</b>.</li> </ul>							

Table 10. Performance of Ultimate SWMF2 under Different Design Storms

Technical Parameter Specifications	Rainfall Storm Events						
	1:2 Year	1:5 Year	1:10 Year	1:25 Year	1:50 Year	1:100 Year	Hazel
<b>PERFORMANCE OF ULTIMATE SWMF2 UNDER 6-HOUR CHICAGO DESIGN STORMS</b>							
Peak inflow to SWMF2 (m <sup>3</sup> /s)	5.31	7.78	9.26	11.10	12.61	14.03	4.49
Peak outflow from SWMF2 (m <sup>3</sup> /s)	0.09	0.24	0.31	0.34	0.35	0.37	4.09
Allowable peak flow* (m <sup>3</sup> /s)	0.14	0.28	0.40	0.58	0.72	0.89	-
Max water level in SWMF2 (m)	266.52	266.67	266.76	266.92	267.03	267.15	267.50
Max storage used in SWMF2 (m <sup>3</sup> )	8,300	11,030	12,820	15,800	17,910	20,300	27,610
Max W/L above the PP (m)	0.52	0.67	0.76	0.92	1.03	1.15	1.50
Max W/depth above the PB (m)	3.52	3.67	3.76	3.92	4.03	4.15	4.50
<b>PERFORMANCE OF ULTIMATE SWMF2 UNDER 24-HOUR SCS TYPE II STORMS</b>							
Peak inflow to SWMF2 (m <sup>3</sup> /s)	2.30	3.24	3.89	4.72	5.41	6.05	-
Peak outflow from SWMF2 (m <sup>3</sup> /s)	0.10	0.30	0.32	0.35	0.36	0.48	-
Allowable peak flow* (m <sup>3</sup> /s)	0.17	0.31	0.43	0.60	0.75	0.90	-
Max water level in SWMF2 (m)	266.59	266.72	266.84	267.00	267.12	267.22	-
Max storage used in SWMF2 (m <sup>3</sup> )	9,520	11,970	14,240	17,270	19,630	21,810	-
Max W/L above the PP (m)	0.59	0.72	0.84	1.00	1.12	1.22	-
Max W/depth above the PB (m)	3.59	3.72	3.84	4.00	4.12	4.22	-
<b>Notes:</b>							
<ul style="list-style-type: none"> <li>The ultimate post-development drainage conditions are illustrated in <b>Figure 8</b>.</li> <li>W/L: water level; Max: maximum; PP: permanent pool; W/depth: water depth; PB: the bottom of the wet pond.</li> <li>*: allowable peak flows are estimated by multiplying the peak flows presented in <b>Table 8</b> by an area reduction factor of <math>24.34/25.67 = 0.95</math> because there is a small area (1.33 ha) out of 25.67 ha draining directly into the Black River rather than into SWMF2 under the ultimate development condition.</li> <li>A hard copy of the interim OTTHYMO output is included in <b>Appendix C</b>.</li> </ul>							

The results of the interim and ultimate hydrologic analysis support the following conclusions:

- Due to the Garfield Wright Blvd extension to Woodbine Avenue and industrial development, quantity of runoff will increase significantly in terms of peak flow and volume under the interim and ultimate conditions. Water quantity control, therefore, is required to alleviate the increase.
- Under the 25-mm design storm, it is expected that the maximum water level may reach 266.4 m within the interim SWMF2. Since over 95% of daily precipitation events in southern Ontario are less than 25 mm, this reflects normal operational conditions of the interim SWMF2.
- It is expected that the maximum water level in the interim SWMF2 will reach 266.5, 266.7 and 267.2 m under the 1:2, 1:5 and 1:100 year Chicago design storms respectively (that is 0.5, 0.7 and 1.2 m above the permanent pool; and 3.5, 3.7 and 4.2 m above the wet pond bottom respectively), in comparison with 266.6, 266.7 and 267.2 m under the 24-hour SCS II design storms respectively. Under the Hurricane Hazel, the maximum water level within the interim SWMF2 can reach as high as 267.5 m.

- Under the interim conditions, the controlled peak outflow from SWMF2 to the Black River is 0.05, 0.18, 0.24, 0.26, 0.27 and 0.29 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year Chicago storms respectively, below the corresponding allowable peak flows of 0.11, 0.21, 0.30, 0.43, 0.54 and 0.67 m<sup>3</sup>/s respectively.
- Under the interim conditions, the controlled peak outflow from SWMF2 to the Black River is 0.06, 0.24, 0.25, 0.27, 0.28 and 0.45 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year SCS II design storms respectively, below the corresponding allowable peak flows of 0.13, 0.24, 0.32, 0.45, 0.56 and 0.68 m<sup>3</sup>/s respectively.
- Under the ultimate conditions, the controlled peak outflow from SWMF2 to the Black River is 0.09, 0.24, 0.31, 0.34, 0.35 and 0.37 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year Chicago storms respectively, below the corresponding allowable peak flows of 0.14, 0.28, 0.40, 0.58, 0.72 and 0.89 m<sup>3</sup>/s respectively.
- Under the ultimate conditions, the controlled peak outflow from SWMF2 to the Black River is 0.10, 0.30, 0.32, 0.35, 0.36 and 0.48 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year SCS II design storms respectively, below the corresponding allowable peak flows of 0.17, 0.31, 0.43, 0.60, 0.75 and 0.90 m<sup>3</sup>/s respectively.
- The results of the hydrologic analysis indicate that the drawdown time of the ultimate SWMF2 is over 42 hours under the 25-mm design storm and satisfies the target minimum drawdown time of 24 hours as required.
- Based on the current design of the ultimate SWMF2, the easement Block 9 (4.2 ha including the area within the regulatory floodplain) is designated for the construction of the ultimate SWMF2, by expanding the interim SWMF2 using same inlet/outlet structures with only minor adjustments. It should be noted that the size and land requirement for the ultimate SWMF2 presented in this Report are subject to the design assumptions made for future development in York Region Industrial Subdivision and the area for the extension of Garfield Wright Blvd.
- As the post-development stormwater runoff from the areas associated with the extension of Garfield Wright Blvd to Woodbine Ave. is to be drained to SWMF2, not toward the intermittent swale as it is now, it is expected that there will be no increase of peak flows at the north limit of York Region Industrial Subdivision under both interim and ultimate conditions.

## 5.0 CONCLUSIONS AND RECOMMENDATIONS

The detailed results of the hydraulic and hydrologic analysis support the following conclusions and recommendations:

1. Because only a small portion of Block 4 is subject to the immediate development in addition to the extension and urbanization of Garfield Wright Blvd to Woodbine Ave, the technical design brief for the interim SWMF2 is presented in this report to accommodate the interim development plan for enhanced water quality, erosion and water quantity controls for a total drainage area of 23.6 ha.
2. The interim SWMF2 is a deep extended wet pond with a sediment forebay, located at the southeast corner of Block 4 and entirely outside of the regulatory floodplain of the Black River. As designed, the interim SWMF2 provides a total storage of 24,884 m<sup>3</sup>, including the permanent pool of 10,066 m<sup>3</sup> and the active storage of 14,818 m<sup>3</sup>. The average depth of the wet pond is 4.60 m, including the 3.0 m permanent pool as requested by the LSRCA and the 1.6 m deep active storage.
3. Under the 25-mm design storm, the maximum water level within the interim SWMF2 may reach 266.4 m approximately. Since over 95% of daily precipitation events in southern Ontario are less than 25 mm, this reflects normal operational conditions of the interim SWMF2.
4. The maximum water level within the interim SWMF2 will reach 266.5, 266.7 and 267.2 m under the 1:2, 1:5 and 1:100 year Chicago storms respectively (that is 0.5, 0.7 and 1.2 m above the permanent pool; and 3.5, 3.7 and 4.2 m above the wet pond bottom respectively), in comparison with 266.6, 266.7 and 267.2 m under the 24-hour SCS II storm events respectively. Under the Hurricane Hazel, the maximum water level within the interim SWMF2 can reach as high as 267.5 m.
5. Under the interim conditions, the controlled peak outflow from SWMF2 is 0.05, 0.18, 0.24, 0.26, 0.27 and 0.29 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year Chicago storms respectively, below the allowable peak flows of 0.11, 0.21, 0.30, 0.43, 0.54 and 0.67 m<sup>3</sup>/s respectively.
6. Under the interim conditions, the controlled peak outflow from SWMF2 is 0.06, 0.24, 0.25, 0.27, 0.28 and 0.45 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year SCS II design storms respectively, below the allowable peak flows of 0.13, 0.24, 0.32, 0.45, 0.56 and 0.68 m<sup>3</sup>/s respectively.
7. Based on the current design, the ultimate SWMF2 needs a total storage of 56,629 m<sup>3</sup>, including the permanent pool of 26,956 m<sup>3</sup> and the active storage of 29,674 m<sup>3</sup> with the average depth of the wet pond of 4.60 m (including the 3.0 m permanent pool and the 1.6 m active storage). As a result, Block 9 (4.2 ha including the area within the regulatory floodplain of the Black River) is reserved and will be

transferred to the Town of East Gwillimbury. The block has been sized to accommodate the ultimate SWMF2, through expanding the interim SWMF2 using the same inlet and outlet structures with minor adjustments to the outlet control structure and emergency overflow weir.

8. Under the ultimate conditions, the controlled peak outflow from SWMF2 is 0.09, 0.24, 0.31, 0.34, 0.35 and 0.37 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year Chicago storms respectively, below the allowable peak flows of 0.14, 0.28, 0.40, 0.58, 0.72 and 0.89 m<sup>3</sup>/s respectively.
9. Under the ultimate conditions, the controlled peak outflow from SWMF2 is 0.10, 0.30, 0.32, 0.35, 0.36 and 0.48 m<sup>3</sup>/s under the 1:2, 1:5, 1:10, 1:25, 1:50 and 1:100 year SCS II design storms respectively, below the allowable peak flows of 0.17, 0.31, 0.43, 0.60, 0.75 and 0.90 m<sup>3</sup>/s respectively.
10. As the post-development stormwater runoff generated from the areas associated with the extension and urbanization of Garfield Wright Blvd to Woodbine Ave is to be drained into SWMF2, not toward the intermittent swale as it is under existing conditions, it is expected that there will be no increase of peak flows at the north limit of the York Region Industrial Subdivision under both interim and ultimate development conditions.
11. It should be noted that the site plans for the future industrial development areas located within Blocks 3 and 4 are not available at this time. When the blocks are developed, they will require site plan approval and the preparation of a storm drainage report to provide a detailed engineering design of the major and minor storm drainage systems and retrofit the interim SWMF2, in accordance with the drainage design criteria outlined in this report, to safely convey runoff into SWMF2 and provide the desired Level 1 (*enhanced*) water quality, erosion and water quantity control for the development.

**Appendix A**

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**Appendix A. Correspondence and Review Comments Received**

Appendix A1: Review Comments Received from the LSRCA Dated June 25, 2004

JUN. 25. 2004 12:53PM LSRCA

NO. 149 P. 1/3



Sent by Facsimile 1-905-763-9983

June 25, 2004

File No.: 19T-94016  
IMS No.: PSDC112C10

Mr. Jaime E. Acosta, P.Eng.  
Cumming Cockburn Limited  
9133 Leslie Street  
Richmond Hill, ON L4B 4N1

Tel: 905-895-1281  
1-800-465-0437  
Fax: 905-853-5881  
E-Mail: [info@lsrca.on.ca](mailto:info@lsrca.on.ca)  
Web: [www.lsrca.on.ca](http://www.lsrca.on.ca)

120 Bayview Parkway  
Box 282  
Newmarket, Ontario  
L3Y 4X1

Dear Mr. Acosta:

Re: **York Region Industrial Subdivision  
Technical Design Brief  
Dated May 2004  
Engineering Drawings  
Dated May 21, 2004  
Part of Lot 2, Concession 4  
Town of East Gwillimbury**

We have completed our review of the above noted submission which we received on May 31, 2004 and comment as follows. These comments are numbered in accordance with those in our letter dated February 2, 2004:

**1.0) Fisheries Comments:**

1.1) The design of the Black River storm outfall channel for Pond SWMF2 is to be revised such that rounded granite rock is used rather than the rip rap which is currently proposed.

1.2) Pond SWMF1 will not be constructed. Pond SWMF3 has already been approved by the Authority. As such, this comment can be considered addressed.

**2.0) Technical Design Brief Comments**

2.1) We will accept the phased approach suggested in the report for the construction of Pond SWMF2.

2.2) Addressed.

**Leaders In**

2.3) Addressed. Please note that the proposed revision to the pond design in your fax dated June 9, 2004 would be acceptable to the Authority. Please revise all related drawings and the report accordingly.

**Watershed**

2.4) The 24 hour event is to be run for the pre-development condition as well. The flows predicted for this event are the target flows for the post-development 24 hour storm scenario.

2.5) Addressed.

**Health**

2.6) Addressed.

2.7) Please provide us with the calculations done to produce CN\*.

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June 25, 2004  
Mr. Jaime Acosta, P.Eng.  
York Industrial Subdivision  
Town of East Gwillimbury  
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2.8) Addressed.

2.9) Addressed.

2.10) Addressed.

2.11) Addressed.

The following are comments arising from our review of the revised SWM brief.

2.12) Page 10. The required water quality storage should be double checked using the total drainage area draining to the pond under the interim condition and the total imperviousness during this interim phase.

2.13) Figure 8. The 8.68 ha. area beside SWMF2 will drain to the pond, not to the river as shown on this figure.

2.14) Table 8. The allowable flows in this table are different than those in Table 9.

2.15) Table 9 shows increases in peak flows (post versus pre) for several of the storm events which is not permissible.

### 3.0 Design Drawing Comments

3.1) Addressed.

3.2) No longer applicable.

3.3) Addressed.

3.4) SWM2 is to show the location of anti-seepage collars on the outlet pipes.

3.5) The landscaping plans have been received by the Authority and are currently under review. Comments will be provided in the near future.

3.6) Notes regarding the pond berm construction are to be added to SWM1.

3.7) Addressed. Due to the infrequent use of the maintenance path, we recommend that it be topsoiled and seeded (on top of the gravel base) in order to provide a more natural boundary to the permanent pool.

3.8) Addressed. Please note that galvanized steel plate orifices are acceptable for use in municipally owned and operated SWM facilities.

June 25, 2004  
Mr. Jaime Acosta, P.Eng.  
York Industrial Subdivision  
Town of East Gwillimbury  
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3.9) Under the 24 hour 1:100 year storm condition, the overflow weir will be used for flow control (0.45 cms). As such, we will require that a concrete weir be installed in the berm as requested previously.

3.10) Addressed.

3.11) Addressed.

3.12) Erosion and Sediment Control Plan Comments

a-f) Due to the limited development proposed during the interim condition, the level of detail on this plan is acceptable.

The silt control fence east of SWMF2 is to be extended past the north limit of the pond.

3.13) Addressed.

3.14) Addressed.

The following are additional comments resulting from our review of the latest design drawings.

3.15) A permit is required for the culvert extensions proposed at station 0+660 on Garfield Wright Blvd. In order to reduce the potential for a HADD, it is suggested that a headwall be fitted on the end of the existing culverts rather than extending them.

3.16) The reference to sodding the pond slopes on drawing SWM1 is to be deleted. Reference should made on this drawing to the pond planting plans.

Should you have any questions regarding the above, please do not hesitate to contact the undersigned. We request that you provide a letter with your next submission detailing how each of the above comments has been addressed. Please refer to the above noted file numbers in all future correspondence.

Yours truly,

Tom Hogenbirk, CMM, P.Eng.  
Manager, Engineering and Technical Services

TH/ph

c Mr. Don Allan, Town of East Gwillimbury, 905-478-2808

S:\TomHEG York Bales LTR.wpd

Appendix A2: Review Comments Received from the Region Dated June 9, 2004



Transportation and Works Department  
Infrastructure Design and Construction  
Fax No. 905-954-4611

June 9, 2004

Mr. Jaime Acosta, P.Eng.  
Cumming Cockburn Limited  
9133 Leslie Street, Suite 200  
Richmond Hill, ON L4B 4N1

Dear Mr. Acosta:

**Re: York Industrial Subdivision  
Storm Sewers  
NE Davis Drive an Woodbine Avenue  
Town of East Gwillimbury, 19T-94016  
York Region Approval No. EG.05.04**

Attached please find the following marked up 1<sup>st</sup> submission documents:

- Partial set of engineering drawings
- Storm sewer design sheets
- Application for Approval of Sewage Works for proposed storm sewers
- Two applications for Approval of Sewage Works for proposed stormwater management pond
- Sample MOE project description for storm sewers and stormwater management pond

In addition, Stormwater Management Report shall be revised to include figures/schematics presenting grading and outlet details for proposed SWM pond under interim and ultimate conditions including description of proposed outlet structure modifications under ultimate conditions. Stage-Storage-Discharge Curves (Table 3 and Table 5) shall be updated to clearly present actual orifice and weir coefficients used in the hydraulic calculations.

Please resubmit revised documents, including Stormwater Management Report to my attention. If you have any questions, please contact me at 905-830-4444 ext. 5749.

Sincerely,

Handwritten signature of Eva Pulnicki.

Eva Pulnicki, M.Eng., P.Eng.  
Environmental Servicing Engineer

EP:in  
Attachments  
Copy to: Don Allan, Town of East Gwillimbury

IDC:\W03\2004\EG-005-04\EG-005-04let\_June09\_04.doc

The Regional Municipality of York, 17250 Yonge Street, Newmarket, Ontario L3Y 6Z1  
Tel: 905-895-1200, 1-877-G04-YORK, Fax: 905-830-6927  
Internet: [www.region.york.on.ca](http://www.region.york.on.ca)



Appendix A3: Review Comments Received from the LSRCA Dated Feb. 2, 2004



Tel: 905-895-1281  
1-800-465-0437  
Fax: 905-853-5881  
E-Mail: [info@lsrca.on.ca](mailto:info@lsrca.on.ca)  
Web: [www.lsrca.on.ca](http://www.lsrca.on.ca)

20 Bayview Parkway  
Box 282  
Newmarket, Ontario  
JY 4X1

Sent By Facsimile 1-905-763-9983

February 2, 2004

File No.: 19T-94016  
IMS No.: PSDC112C4

Mr. Kevin Walters, P.Eng.  
Cumming Cockburn Limited  
9133 Leslie St., Ste. 200  
Richmond Hill, ON L4B 4N1

Dear Mr. Walters:

Re: **York Region Industrial Subdivision  
Technical Design Brief  
Dated November 19/03  
Engineering Drawings  
Dated November 1/03  
Part of Lot 2, Concession 4  
Town of East Gwillimbury**

We have completed our review of the aforementioned design brief (received November 24/03) and engineering drawings (received December 4/03). Please be advised that effective March 1, 1998 the Board of Directors of the Conservation Authority adopted Staff Report 3-98-BOD which provided for the collection of fees for the review of planning and engineering submissions to the Conservation Authority. As such, we will require a review fee in the amount of \$2,500.00 for this development. Please remit this payment with your next submission.

**1.0) Fisheries Comments**

As part of our Level III agreement with Department of Fisheries and Oceans (DFO), our fisheries biologist, Jeff Anderson, has reviewed the above noted submission and provides the following comments:

1.1) Additional details are required for the outfall from SWMF2 into the Black River.

1.2) SWM F1 and SWM F3, are noted as discharging to or being placed on line of an "Intermittent Swale". This swale was electrofished approximately 600 metres downstream of the subject property by the Ontario Ministry of Natural Resources (OMNR) in 1995. This effort exposed the presence of blacknose dace (*Rhynchichthys atratulus*), northern redbelly dace (*Phoxinus eos*) and mottled sculpin (*Cottus bairdi*). The presence of mottled sculpin is significant as they are a coldwater fish that are indicators of groundwater activity.

Due to this, it is suggested that the intermittent swale represents fish habitat and an on-line pond would represent a HADD.

The DFO could support off-line treatment draining to this watercourse if stream temperatures were respected and maintained or enhanced (e.g. riparian plantings).

We would be pleased to meet with you on this matter at your earliest convenience.

Page 1 of 4

*Leaders In*

*Watershed*

*Health*





February 2, 2004  
Mr. Kevin Walters, P.Eng.  
York Region Industrial Subdivision  
File No.: 19T-94016  
IMS No.: PSDC112C4  
Town of East Gwillimbury  
Page 2 of 4

## 2.0) Technical Design Brief Comments

2.1) In general, the Authority prefers that SWM facilities be built out to their ultimate design capacity in order to minimize disturbance in the future. It also would simplify the development of the remaining lots in the subdivision as the builder on these lots would not need to perform any off site works. It is recommended that the design of the ponds be revised accordingly.

2.2) If addressing the above noted comment is not feasible at the present time, we will require the following revisions in the report.

a) A separate section is required on the ultimate design parameters for ponds F2 and F3 including total upstream areas and assumed imperviousness. This section should include a figure showing the extent of the fully developed drainage areas to be serviced by the ponds along with the final pond sizes.

b) Figure 5 is to be revised to clearly identify the areas (developed and undeveloped) that will be serviced by the interim ponds.

c) Figure 5 should include dashed-in outlines of the estimated ultimate foot prints of SWM F2 and F3. There must be sufficient land set aside for the expansion of these ponds in the future.

2.3) As the Black River and its tributaries in this area are considered coldwater, we will require that the SWM ponds be designed as deep wet ponds (3 metres) with bottom draws. The shape of the pond should be designed in such a manner to facilitate shading by bank vegetation.

2.4) The quantity control volumes for the SWM ponds are to be checked using the 24 hour SCS storm distributions for the various storm events.

2.5) A disk of the OTTHYMO input / output files is to be provided.

2.6) The hydrologic modelling for the site is to include the modelling completed for Blocks 1 and 2 by Marshall Macklin Monaghan. The modelling needs to demonstrate that there will not be an increase in peak flows at the north limit of the subdivision.

2.7) Information is to be provided justifying the use of 75 as the curve number for the subject property.

2.8) A table should be provide listing the input parameters for each of the catchment areas (pre and post development) and how these were derived.

2.9) Draw down calculations (as per the equations in the 2003 SWMPP Design Manual) are required to demonstrate that 24 hour detention is provided for the run off from a 25 mm event.

2.10) Emergency overflow weirs and supporting calculations are required for the SWM facilities.

2.11) The report should include a section on the operation and maintenance requirements for the SWM pond including standard operation methods, recommended inspection program, expected frequency of forebay clean outs and recommended methodology for removal of sediment, grass cutting and weed control.



February 2, 2004  
Mr. Kevin Walters, P.Eng.  
York Region Industrial Subdivision  
File No.: 19T-94016  
IMS No.: PSDC112C4  
Town of East Gwillimbury  
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### 3.0) Design Drawing Comments

- 3.1) Emergency overflow weirs are required on the SWM ponds.
- 3.2) Please verify the sizing of the outlet swale from Street B (1:100 year?) The rip rap in this swale should be underlain with filter fabric.
- 3.3) The lowest part of the reverse sloped pipes are to be anchored securely.
- 3.4) Anti seepage collars are to be provided on the outlet pipes from all SWM facilities.
- 3.5) A detailed landscaping plan is required for each of the SWM ponds. Please note that a topsoil thickness of 300 mm is required through all pond areas and that creeping red fescue should not be included in ground cover seed mixtures specified for the ponds. Native species of trees and shrubs should be used. In addition, shade trees should be planted near the flood fringe around the permanent pool and thicker groupings of these should be concentrated at the south and west ends of the pond.
- 3.6) Notes on the construction of the pond berms (i.e. acceptable soils with low permeability to be used, inspection by a geo-tech and compaction %) are to be included on the SWM pond drawings.
- 3.7) The proposed 4.0 metre wide gravel access road around the permanent pools in the SWM ponds would inhibit the pond's ability to naturalize. In addition, it would make it more difficult to shade the water using trees and shrubs on the banks. As such, we will require that the extent of these roads be reduced to a minimum.
- 3.8) Details are required for the proposed outlet control structures.
- 3.9) As it is proposed to use the weir structures as flow control devices for the less frequent events, we will require that a concrete weir be installed in the berm to more precisely set the weir shape and elevation. A section should be provided detailing the shape of each weir.
- 3.10) The clay liner in the SWM pond should be extended to above the permanent pool water level.
- 3.11) The plan for SWMF2 should show the regional flood line (265.13 masl).
- 3.12) Erosion and Sediment Control Plan Comments:  
The Sediment and Erosion Control Plan must include the following:



February 2, 2004  
Mr. Kevin Walters, P.Eng.  
York Region Industrial Subdivision  
File No.: 19T-94016  
IMS No.: PSDC112C4  
Town of East Gwillimbury  
Page 4 of 4

- a) Topsoil stockpile locations.
  - b) Stone mud mats at all construction entrances.
  - c) The SWM ponds should be temporarily fitted with filter fabric / clear stone wrapped Hickenbottom riser outfalls (with anti seepage collars) and rip rap overflow weirs. The riser should be surrounded by stone and this stone wrapped in filter fabric. A final layer of stone should then be placed on the filter fabric. This substantially increases the fabric surface area and thus reduces the potential for clogging. These ponds are to be sized to provide a minimum of 125 m<sup>3</sup>/ha. 24 hr. extended detention and 125 m<sup>3</sup>/ha. "permanent" pool storage. Larger ponds may be required depending on soil type and erosion potential.
  - d) Notes on the installation timing, inspection and maintenance of sediment controls. Sediment controls must be inspected on a regular basis and after every rain fall event. Repairs must be done in a timely manner to prevent movement of sediment into the water.
  - e) Lines delineating the limit of cut and fill areas
  - f) Notes requiring the stabilization of all areas which will remain disturbed for more than thirty days.
- 3.12) The storm drainage plan (STM1) appears to delineate only the drainage areas tributary to the road storm sewers. This should be clearly stated on this plan.
- 3.13) The drainage areas for the developed portions of Block 4 are labelled but not delineated on plan STM1.
- 3.14) The legend for plan STM1 has incorrect labels for the drainage catchment areas.

Should you have any questions regarding the above, please do not hesitate to contact the undersigned. Please reference the above file numbers in all future correspondence.

Yours truly,

Tom Hogenbirk, P.Eng.  
Manager, Engineering and Technical Services

TH/ph

c Mr. Don Allan, Town of East Gwillimbury, 905-478-2808

S:\Tom\H&E\York Bates LTR.wpd

Appendix A4: Review Comments Received from the LSRCA Dated April 16, 2004

APR. 16. 2004 3:21PM

LSRCA

NO. 427

Sent By Facsimile 1-905-763-9983



April 16, 2004

File No.: 19T-94016  
IMS No.: PSDC112C7

Mr. David Bradley, P.Eng.  
Cumming Cockburn Limited  
9133 Leslie Street  
Richmond Hill, ON L4B 4N1

Tel: 905-895-1281  
1-800-465-0437  
Fax: 905-853-5881  
E-Mail: [info@lsrca.on.ca](mailto:info@lsrca.on.ca)  
Web: [www.lsrca.on.ca](http://www.lsrca.on.ca)

Dear Mr. Bradley:

120 Bayview Parkway  
Box 282  
Newmarket, Ontario  
L3Y 4X1

Re: **York Region Industrial Subdivision  
Technical Design Brief  
Dated November 19/03  
Engineering Drawings  
Dated November 1/03  
Part of Lot 2, Concession 4  
Town of East Gwillimbury**

Further to our previous letter dated February 2, 2004, staff of the Authority visited the site on April 16, 2004. The results of this site visit, which was conducted by our Senior Fisheries Biologist and our Aquatic Ecologist, are summarized as follows:

The noted intermittent "swale" was flowing the day of investigation. Fish were found at a culvert approximately 188 metres downstream of Bales Drive wherein Brook Stickleback (*Culaea inconstans*) were observed and captured. Upstream of Bales Drive the tributary flowed through a small wetland (west of the pipe under Bales Drive) and continued upstream to cross Bales Drive second time (see attached Map).

Regardless of existing development, this "swale" represents the headwaters of a fully functioning cold to coolwater tributary. The waters of this tributary would not require stormwater treatment.

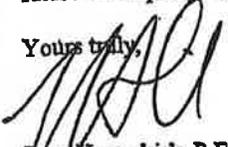
For these reasons, Authority staff can not support the proposed on-line pond as it would constitute a HADD. We would fully support a properly designed off-line facility that would take stormwater flows from both Bales Drive and the eastern swale.

Should you have any questions regarding the above, please do not hesitate to contact the undersigned or Jeff Andersen of our office. Please refer to the above noted file numbers in all future correspondence.

*Leaders In*

Yours truly,

*Watershed*

  
Tom Hogenbirk, P.Eng.  
Manager, Engineering and Technical Services

*Health*

TH/ph

c Mr. Don Allan, Town of East Gwillimbury, 905-478-2808

S:\TomH\EG York Bales LTR.wpd



Appendix A5: CCL Letter to the LSRCA on Revised SWM Plan Dated May 14, 2004



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consulting engineering | planning | environmental approvals

File: 5390-10

May 14, 2004

Lake Simcoe Region Conservation Authority  
Engineering and Technical Services  
120 Bayview Parkway, Box 282  
Newmarket, ON L3Y 4X1

**ATTENTION: Mr. Tom Hogenbirk, P. Eng.  
Manager, Engineering and Technical Services**

Dear Sir:

**Re: Revised Drainage Areas for the Stormwater Management Plan  
York Region Industrial Subdivision 19T-94016  
Northeast of Woodbine Avenue and Davis Drive  
Town of East Gwillimbury, Regional Municipality of York**

In accordance with our telephone conversation on May 3, 2004, enclosed please find the revised drainage areas for the stormwater management plan of the York Region Industrial Subdivision.

The intent of the revised stormwater management plan is to maximize useable lands, minimize the number of the stormwater management facilities, and maintain the existing drainage pattern to the intermittent swale as much as possible for the existing fish habitat.

Under the revised stormwater management plan, previously proposed facilities SWMF4 and SWMF5 (located at the northwest and northeast corners of the intersection of Garfield Wright Boulevard and the swale respectively) are eliminated, and all the developments within Blocks 3 and 4 and the areas associated with the expansion and urbanization of Garfield Wright Blvd will be accommodated by SWMF2 located at the southeast corner of Block 4.

Since the Authority has classified the intermittent swale as the headwater of a fully functioning cold to coolwater tributary that would not require stormwater treatment regardless the existing development, the current drainage conditions within the existing Bales Industrial Subdivision and the majority of the area to its west will remain and continue draining toward the intermittent swale as the source of water supply for the existing fish habitat. Only the

J:\5000\5390-York Industrial East Gwill10-SWM\CORRESP\Letter to LSRCA on May 14 2004.doc



Consulting  
Engineers  
of Ontario

9133 Leslie Street, Suite 200 Richmond Hill, Ontario L4B 4N1 | T: 905-763-2322 | F: 905-763-9983



minimum area associated with the immediate development for the expansion and urbanization of Garfield Wright Boulevard will be drained into SWMF2 for erosion, water quality and water quantity control. Since the extension part of Garfield Wright Boulevard and the small area to its south belong to the new development, the peak flows up to the 1:100 year design storm from this area will be conveyed through the storm sewer system into SWMF2 for water quantity control.

Since only a part of Blocks 3 and 4 is subject to the immediate industrial development in addition to the expansion and urbanization of Garfield Wright Blvd, SWMF2 will be designed to accommodate the interim development, but one stormwater management block will be reserved and designated for the future expansion or retrofit of SWMF2 to accommodate the ultimate development of the subdivision.

Based on the assumptions mentioned above, we are proceeding with the detailed engineering design of SWMF2 and the extension and urbanization of Garfield Wright Blvd and Bales Drive East, and addressing your comments expressed in the letters dated Feb. 2 and April 16 of 2004. Should you have any question or concern with regard to the revised stormwater management plan, please do not hesitate to contact the undersigned.

Yours very truly,

**CUMMING COCKBURN LIMITED**

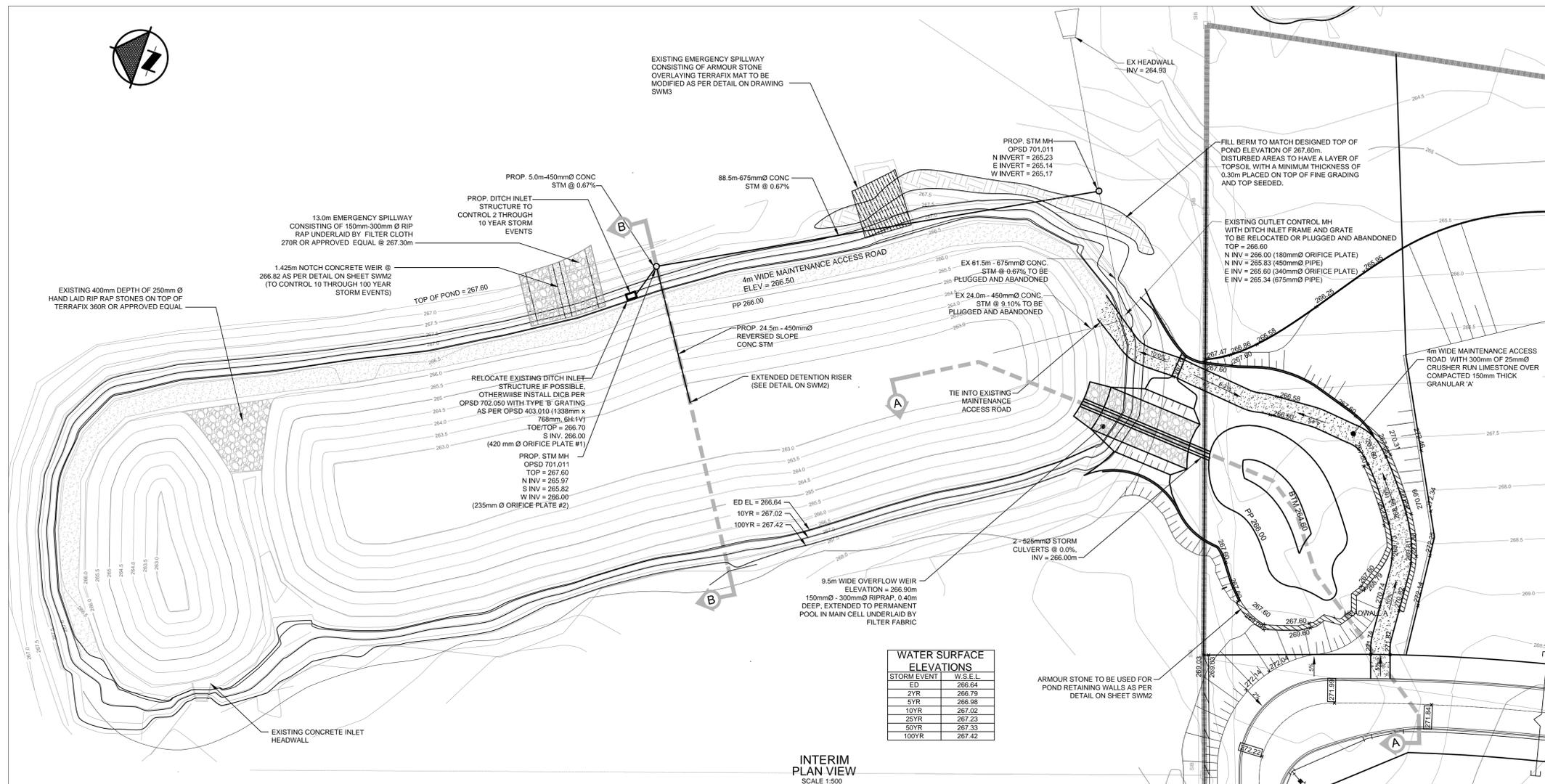


Nicky Chang

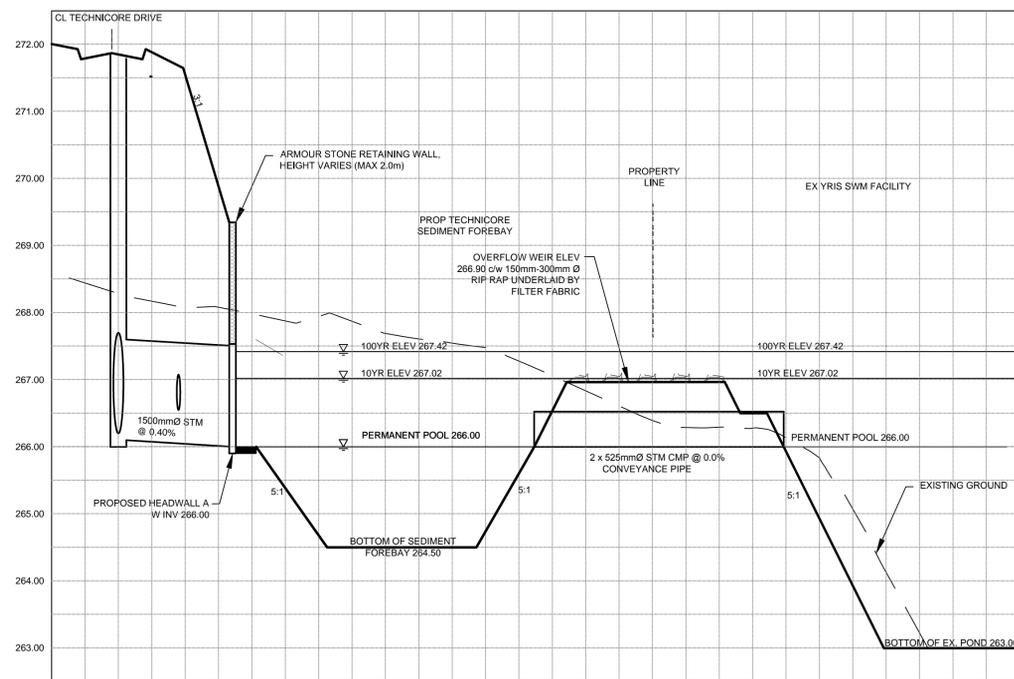
cc: Marvin Finkel, Matrix Management Corp.  
Barry Crowe, Regional Municipality of York  
Don Allan, Town of East Gwillimbury  
Paul Vincent, URS-Cole Sherman  
Richard Knight, Marshal Macklin Monaghan Limited

JB/nc

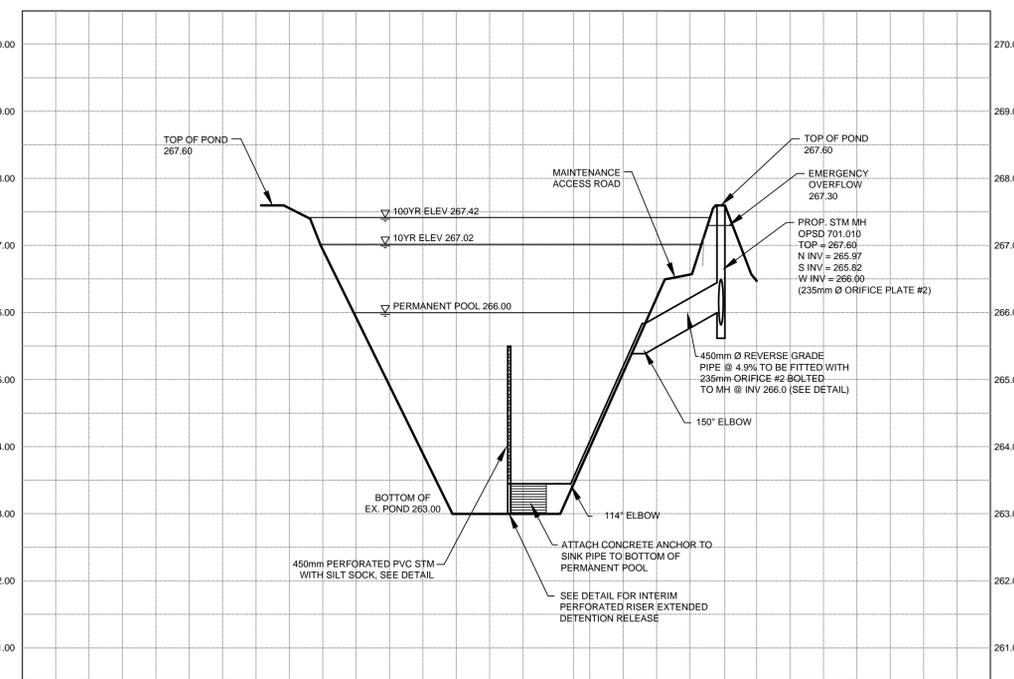




INTERIM PLAN VIEW  
SCALE 1:500



INTERIM SECTION A-A  
SCALE H 1:500 V 1:50



INTERIM SECTION B-B  
SCALE H 1:500 V 1:50

**LEGEND**

- WM EXISTING WATERMAIN
- SAN EXISTING SANITARY SEWER
- STM EXISTING STORM SEWER
- CB EXISTING CATCHBASIN LEAD
- GAS EXISTING GAS MAIN
- BELL EXISTING U/G BELL
- HYDRO EXISTING U/G HYDRO
- CATV EXISTING U/G CABLE TELEVISION
- BELL EXISTING BELL PEDESTAL
- HP BP P EXISTING HYDRO TRANSFORMER & PAD
- HP EXISTING UTILITY POLE WITH GUY WIRE
- HYD EXISTING HYDRANT AND VALVE
- VC EXISTING VALVE CHAMBER
- WB EXISTING VALVE & BOX
- GV EXISTING GAS VALVE
- MH EXISTING MH STORM OR SANITARY
- CB EXISTING CATCHBASIN SINGLE
- DCB EXISTING CATCHBASIN DOUBLE
- IRB IRON BAR
- IRB STANDARD IRON BAR
- CG EXISTING CURB & GUTTER
- CL EXISTING CULVERT
- P&W CL P&B EXISTING FENCE
- DT EXISTING DECIDUOUS TREE
- CT EXISTING CONIFEROUS TREE
- VEG EXISTING VEGETATION TO BE REMOVED
- SIGN EXISTING SIGN
- MH1 PROPOSED STORM MAINTENANCE HOLES
- CBMH1 PROPOSED CATCHBASIN MAINTENANCE HOLES
- MH1 PROPOSED SANITARY MAINTENANCE HOLES
- PSW PROPOSED SEWER
- PVC W/M POSSIBLE PIPE DISCHARGE TO EXISTING DITCH
- BCB PROPOSED WATERMAIN
- CB PROPOSED CATCHBASIN
- DCB PROPOSED DOUBLE CATCHBASIN
- VC PROPOSED VALVE & CHAMBER
- CG EXISTING CURB & GUTTER & DEPRESSION
- SWW PROPOSED SIDEWALK
- GRAV RESTORE DRIVEWAY WITH GRAVEL
- ASP RESTORE DRIVEWAY WITH ASPHALT
- CONC RESTORE DRIVEWAY WITH CONCRETE
- P&S RESTORE DRIVEWAY WITH PAVING STONE
- DRAP DIRECTION OF DRIVEWAY APRON DRAINAGE
- ELEV PROPOSED ELEVATION
- ELEV EXISTING ELEVATION
- STR EXISTING STRUCTURE TO BE REMOVED
- EX SAN EXISTING SAN LATERAL CONNECTIONS
- CONN CONNECTIONS TO EXISTING WATER SERVICE AT PROP. AND EX. WATERMAINS

No.	Revision	Date	By	Appr'd
1	1ST ENGINEERING SUBMISSION	JULY 24/13	FJB	

No.	Elevation	Description
1.	263.981m	ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE DERIVED FROM MINISTRY OF TRANSPORTATION BENCHMARK NO. 0819168475. LOCATED ON THE WEST SIDE OF WOODBINE AVENUE ON A 2-STORY FRAME HOUSE WITH ALLUMBIUM SIDING. 1.1 KM SOUTH OF THE JUNCTION OF DAVIS DRIVE AND WOODBINE AVENUE.

CONSULTANT

TOWN OF E.G. ENGINEERING DEPT

SIGNATURE \_\_\_\_\_ DATE \_\_\_\_\_

**BURNSIDE**

R.J. Burnside & Associates Limited  
6990 Creditview Road, Unit 2  
Mississauga, Ontario, L5N 8R5  
Telephone (905) 821-1800  
Fax (905) 621-1809  
Web www.burnside.com

**TECHNICORE INDUSTRIAL SUBDIVISION**

**STORMWATER MANAGEMENT FACILITY**

Surveyed by: RA	Checked by: FB	Project No. PID018673
Drawn by: RA	Approved by: FB	Drawing No. SWM1
Designed by: DT	Date: NOV. 28, 2012	Sheet No. 8 OF 12

Scale: 1:400

Date Plotted: 01/21/2013 09:05:24 AM

# APPENDIX B

**Sanitary Demand Calculations**

**Percolation Test Report**

**Septic System Design Information**

**Sanitary Design Calculations**

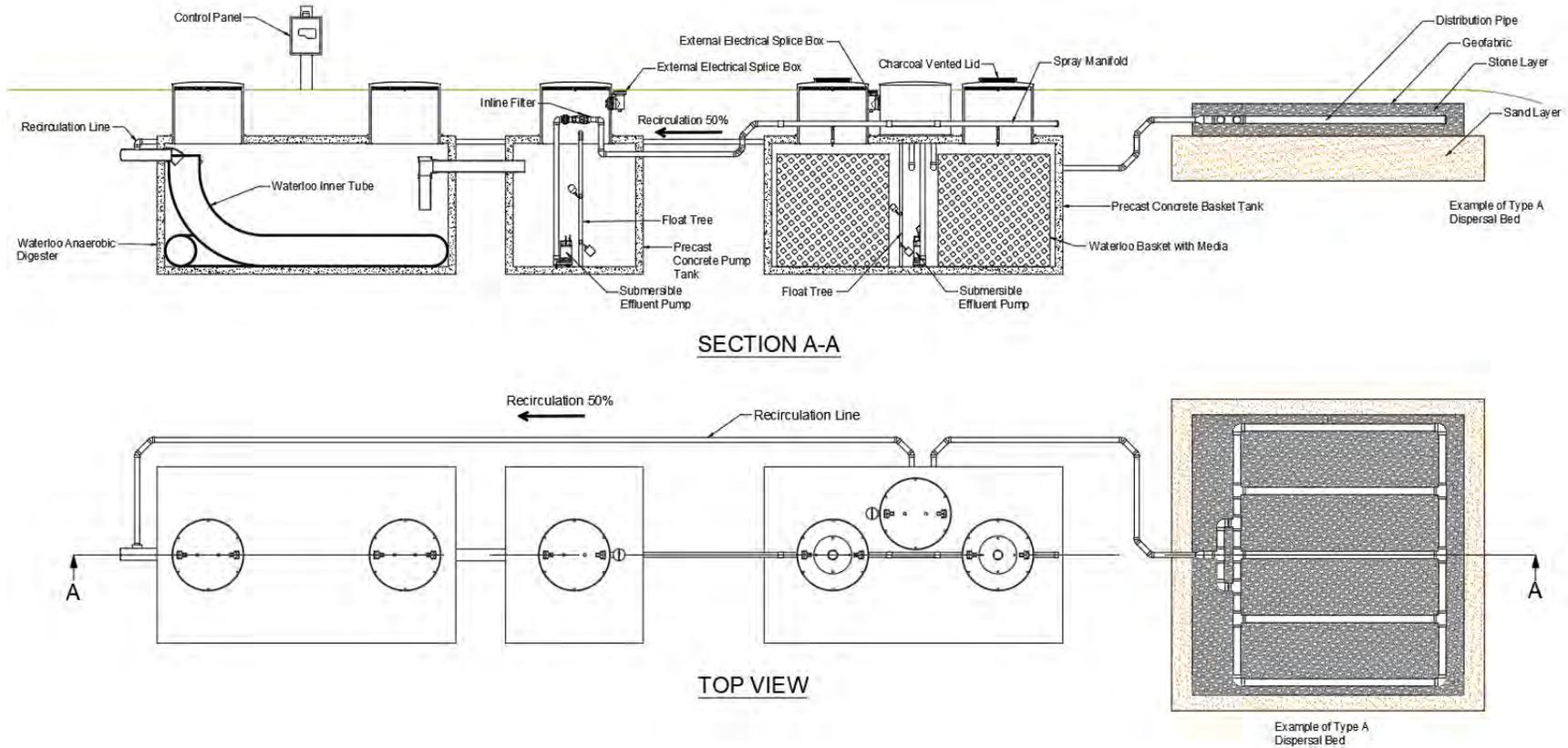
**Project:** 350 Garfield Wright Boulevard  
**Project No:** 24015  
**Client:** York Regional Police  
**Location:** East Gwillmbury, Ontario  
**Site Area:** 0.67 ha (development area only)  
**Date:** 29-Aug-24

---

**Daily Sanitary Design Flow**

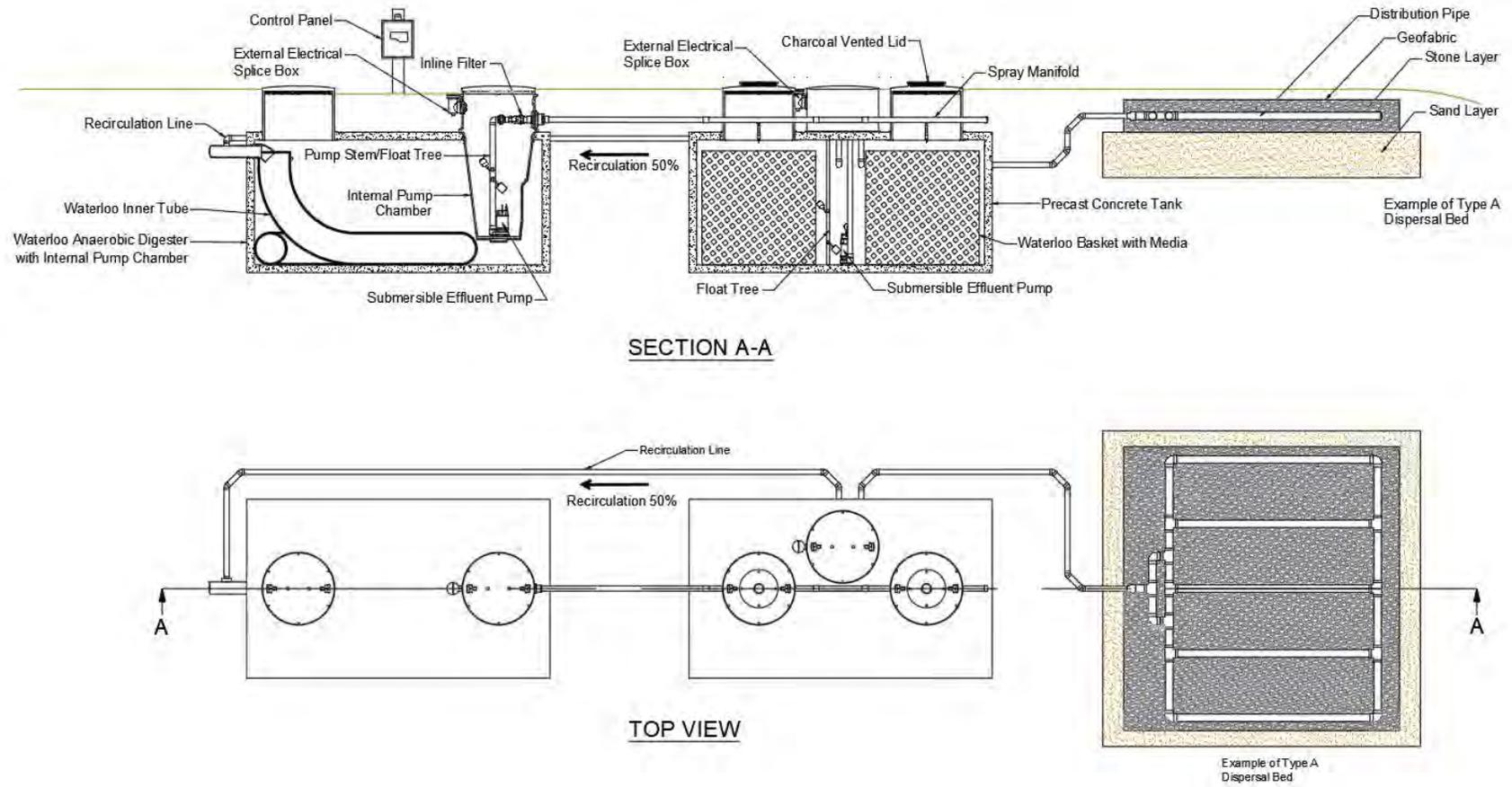
Ontario Building Code Non-Residential Design Flow Rates				
Occupancy	Unit	Daily Volume, Litres per unit *	Site Units	Daily Design Volume (Litres)
<b>Office Building</b>				
Per each 9.3 m <sup>2</sup> of floor space	9.3 sq.m	75	450	3,629
Per 2012 OBC Code, Table 8.2.1.3.B		Average Flow =		0.04 L/s
		=		2.52 L/min

## System Diagram - Baskets in Concrete Tank

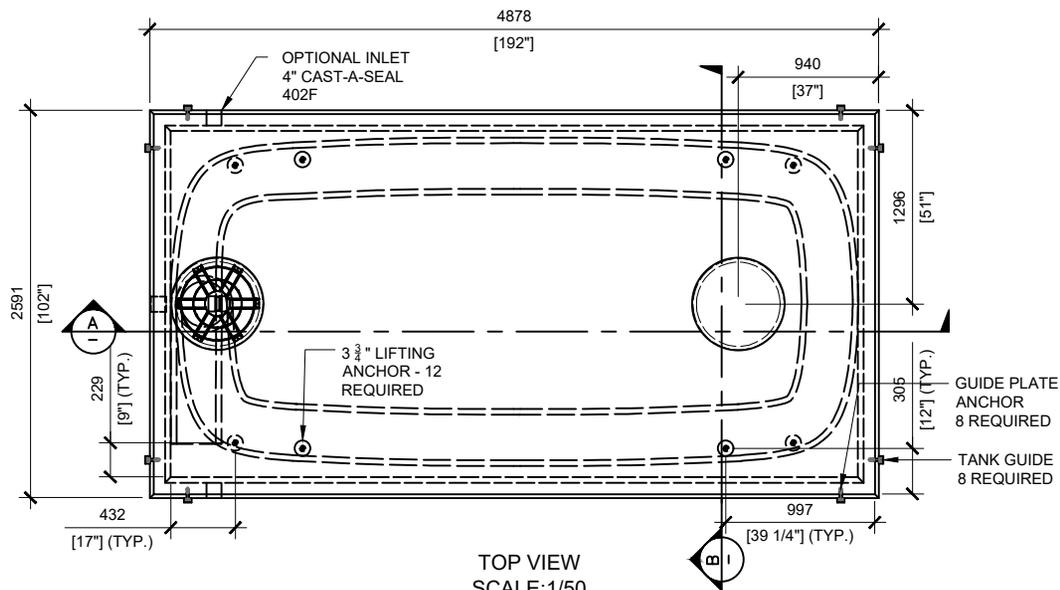


**Figure 51.** Anaerobic digester, pump tank, and baskets in concrete tank system diagram

## System Diagram - Baskets in Concrete Tank

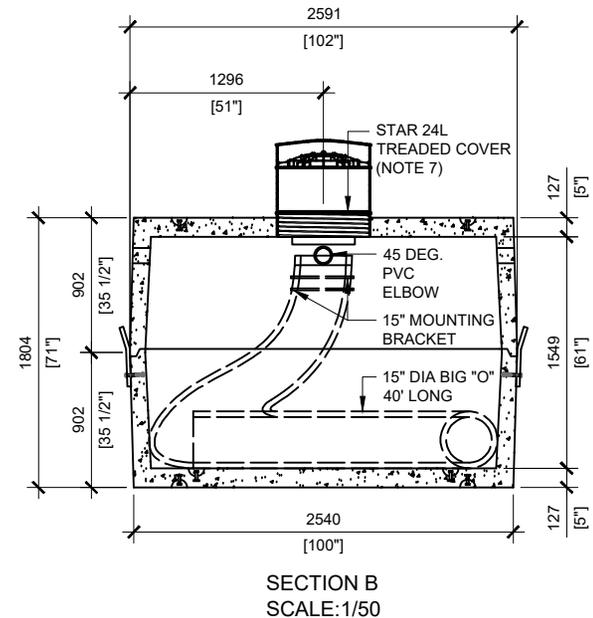
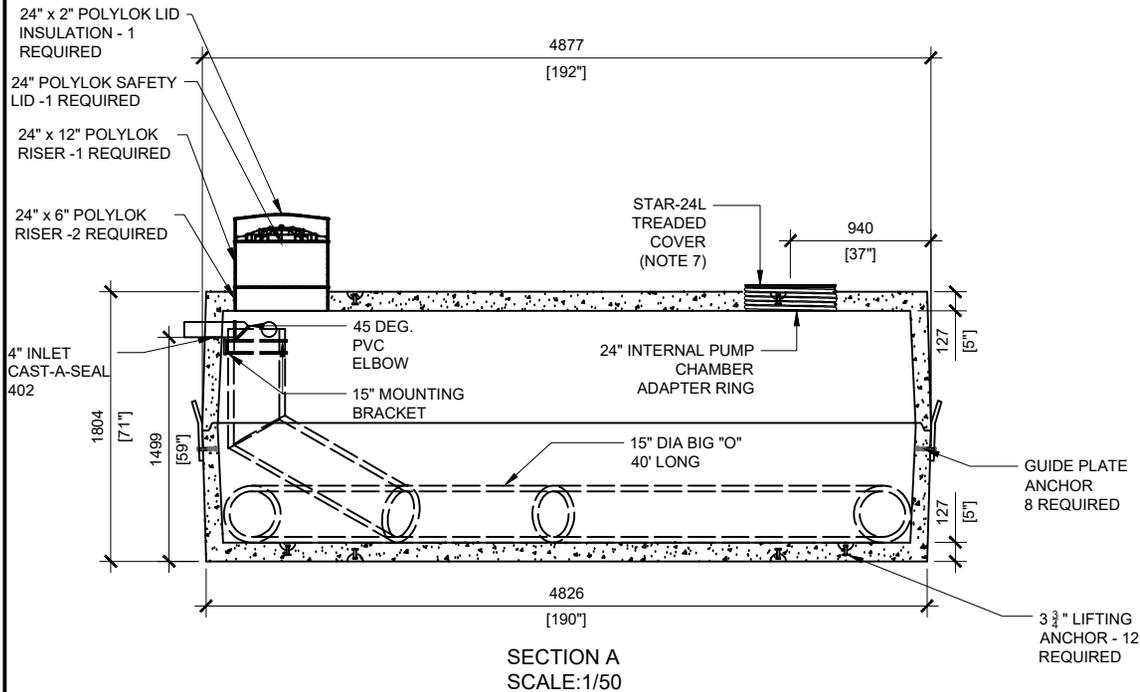


**Figure 50.** Anaerobic digester with internal pump chamber and baskets in concrete tank system diagram



**GENERAL NOTES:**

1. UNITS ARE SEALED WITH BUTYL TAPE AT THE JOINTS
2. DELIVERY IS MADE BY CRANE-EQUIPPED TRUCKS
3. EXCAVATION MUST BE READY, SAFE AND ACCESSIBLE FOR UNLOADING FROM THE REAR OF THE TRUCK.
4. MIN OVERHEAD CLEARANCE OF 18FT (5.5 METRES) IS REQUIRED
5. ALL UNITS MUST BE HANDLED WITH PROPER LIFTING EQUIPMENT (I.E. SPREADER BAR)
6. MAXIMUM BURIAL DEPTH = 1 METRE IN FIRM SOIL AWAY FROM ANY VEHICULAR TRAFFIC
7. THREADED COVER TO BE REMOVED AND REPLACED BY INTERNAL PUMP CHAMBER (PROVIDED BY WATERLOO) ON SITE



**MANUFACTURED:**  
LINDSAY, ON  
1-800-655-3430

**CONCRETE TYPE:** SCC  
**CONCRETE:** 45MPa at 28 days / 6,500PSI  
**AIR ENTRAINMENT:** 5-8%  
**REINFORCEMENT:** STEEL TO CSA CAN  
A23.1 / A23.3 G30.18 Fy=400MPa

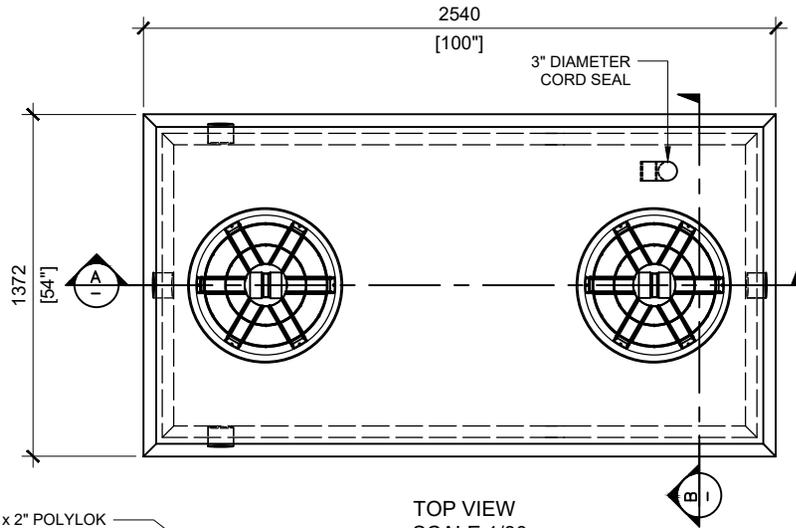
**WEIGHT:**  
BOTTOM - 17,712lbs / 8,050kg  
TOP - 17,571lbs / 7,989kg

**DRAWN BY:**  
PRASHAN

**DATE:**  
DEC/2023

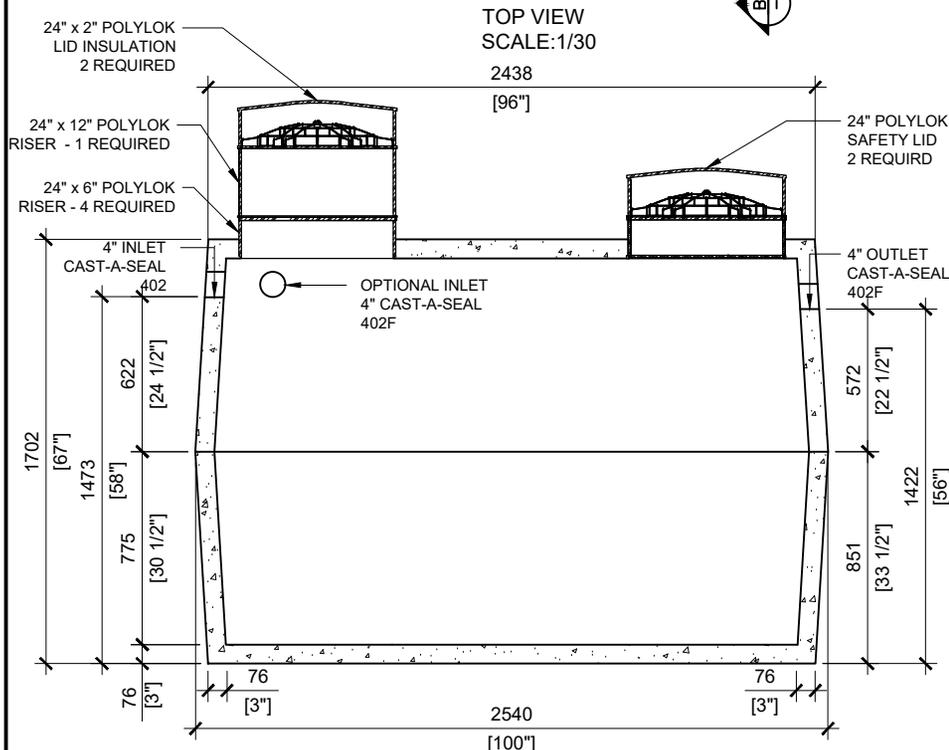
**WATERLOO ADIPC-14000**

14,000 LITRES

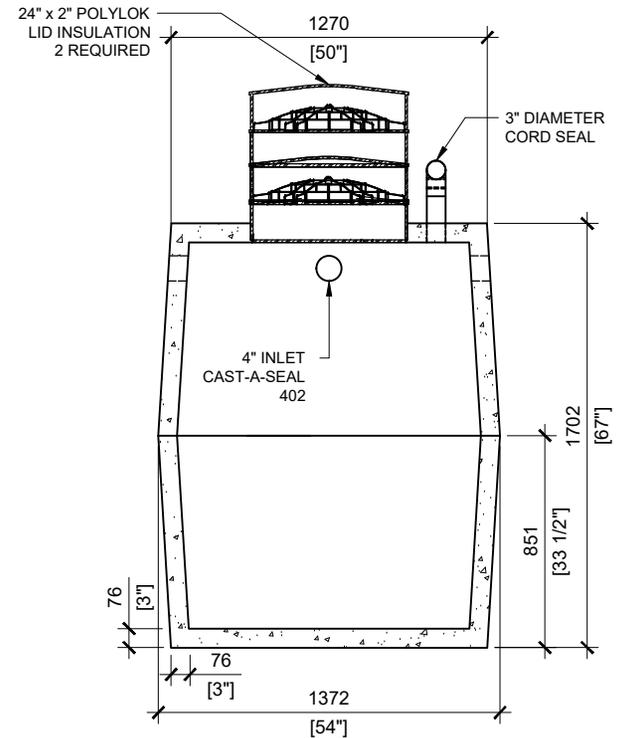


**GENERAL NOTES:**

1. UNITS ARE SEALED WITH "NON-TOXIC" BUTYL TAPE AT THE JOINTS
2. DELIVERY IS MADE BY CRANE-EQUIPPED TRUCKS
3. EXCAVATION MUST BE READY, SAFE AND ACCESSIBLE FOR UNLOADING FROM THE REAR OF THE TRUCK.
4. MIN OVERHEAD CLEARANCE OF 18FT (5.5 METRES) IS REQUIRED
5. ALL UNITS MUST BE HANDLED WITH PROPER LIFTING EQUIPMENT
6. MAXIMUM BURIAL DEPTH = 1 METRE IN FIRM SOIL AWAY FROM ANY VEHICULAR TRAFFIC
7. POLYLOK SAFETY LIDS INSTALLED IN BOTH OPENINGS AS PER CSA-B66-21
8. 24"x12" POLYLOK RISER WITH ELECTRICAL SPLICE BOX SUPPLIED BY WATERLOO AT THE OUTLET



SECTION A  
SCALE: 1/30



SECTION B  
SCALE: 1/30



**MANUFACTURED:**  
LINDSAY, ON  
1-800-655-3430

**CONCRETE:** 35MPa at 28 days / 5000PSI  
**AIR ENTRAINMENT:** 5-8%  
**REINFORCEMENT:** STEEL TO CSA CAN  
A23.1 /A23.3 G30.18 Fy=400MPa

**UNIT WEIGHT:** 6,520lbs / 2,963kg

**DRAWN BY:**  
PRASHAN

**DATE:**  
FEB/2024

**WATERLOO PT-3600**

3,600 LITRES



July 31, 2024

Azimuth Environmental Consulting Inc.  
642 Welham Road  
Barrie, Ontario  
L4N 9A1

Attn: Brendan MacNaughton

**RE: Job No. 24-054**  
**Determination of Estimated T-Time**

GEI Consultants Ltd. (GEI) was provided with three (3) soil samples on July 23, 2024 to complete grain size analyses to determine the percolation rate of the tested soils (T-Time analysis).

The delivered samples were identified as shown below.

- TP-24-1-2, YRP Hanger
- TP-24-6-4, YRP Hanger
- TP-24-3-2, YRP Hanger

Three grain size distribution curves were developed by testing the above referenced soil samples in accordance with ASTM D6913 Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis and ASTM D7928 (sedimentation / hydrometer analysis). The result of the laboratory test and graphical representation of the grain size analyses are enclosed.

Determination of percolation rate is based on the “*Ministry of Municipal Affairs and Housing (MMAH) Supplementary Guidelines SB-6, Percolation Time and Soil Descriptions, September 14, 2012*”. Based on this document, a summary of the result and the estimated percolation rates of the soil are as follows:

Client Reference	Soil Description (MIT)	USCS Soil Classification	Coefficient of Permeability (K- cm/sec)	Estimated Percolation Rate or “T-Time” (mins/cm)
TP-24-1-2	SILT, Some Clay, Trace Sand	M.L.	$<10^{-6}$	>50 mins/cm
TP-24-6-4	SILT, Some Sand, Some Clay, Trace Gravel	M.L.	$10^{-6}$	50 mins/cm
TP-24-3-2	SANDY SILT, Some Clay, Trace Gravel	M.L.	$10^{-6}$	50 mins/cm

\*Reference MMAH Supplementary Standard SB-6, Table 2

It is noted that percolation time not only varies based on the grain size distribution but is also influenced by other soil characteristics such as the density of the soil, the structure of the soil, the percentage/mineralogy of clay, the plasticity of the soil, the organic content of the soil, and the groundwater table level which are not expressly calculated as part of a grain size analysis.

No field investigation was conducted by GEI in conjunction with the above testing and did not witness the depth or location in which these samples were obtained. GEI is providing the percolation rates as factual information, to be used in design by a qualified professional with due regard to the limitations as indicated above.

We trust this information is sufficient for your present purposes. Should you have any questions concerning the above, or if we can be of any further assistance, please do not hesitate to contact the undersigned.

Yours truly,  
**GEI Consultants Ltd.**



Donna Davidson-Gorry  
Laboratory Supervisor  
(705) 718-6604  
ddavidsongorry@geiconsultants.com



Andrew Jones  
Materials Testing and Inspection Practice Lead  
(705) 220-0060  
ajones@geiconsultants.com

Enclosures (3)

Grain Size Analysis (T-Time)

## **ENCLOSURE 1**

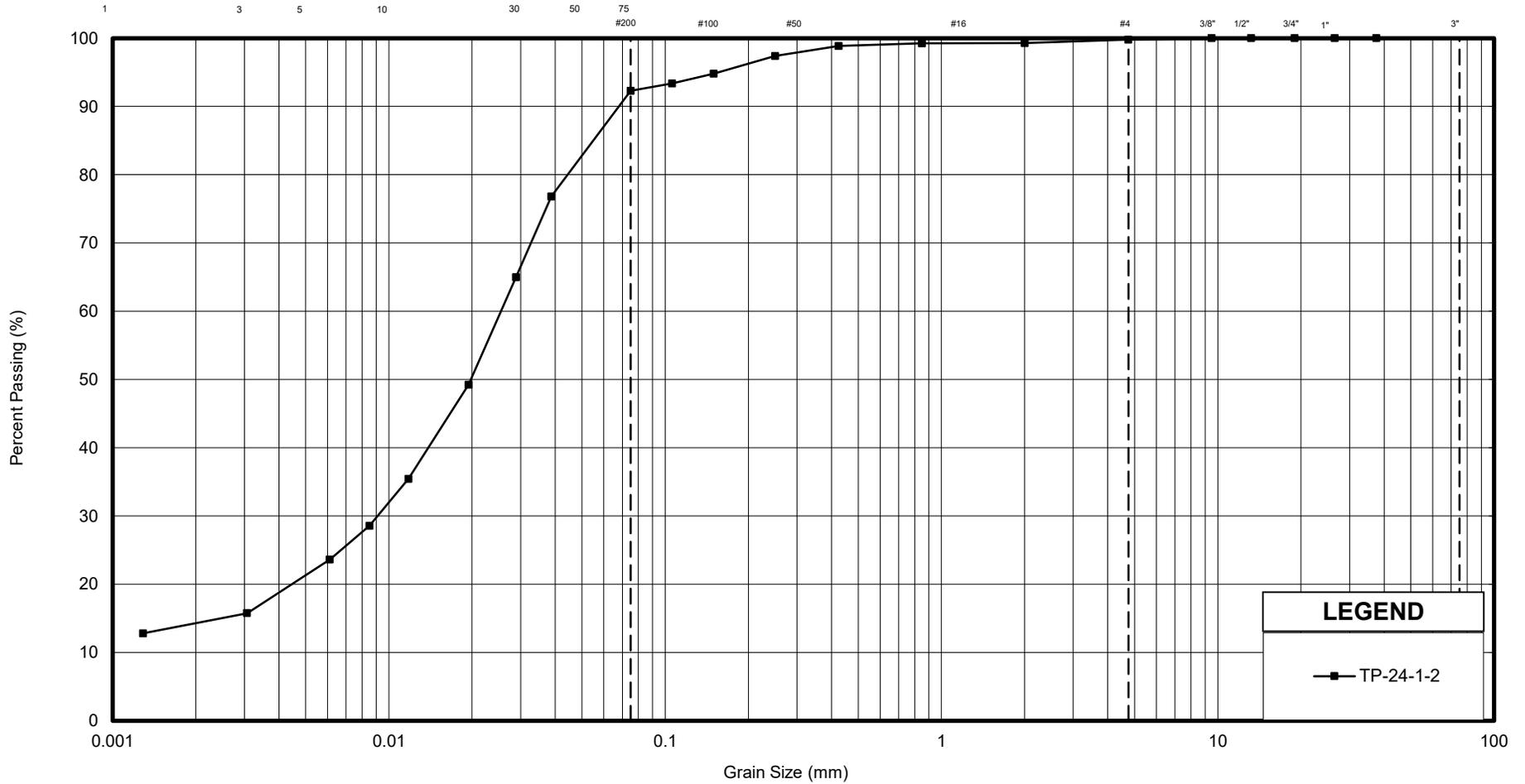
Grain Size Analysis (T-Time)

**UNIFIED SOIL CLASSIFICATION SYSTEM**

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (IMPERIAL)



**LEGEND**

—■— TP-24-1-2

GEI Lab No.	Description	Gr.	Sa.	Si.	Cl.	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>
7836	SILT, Some Clay, Trace Sand	-	7	78	15	-	0.009	0.025	-	-



Azimuth Environmental - Job No. 24-054, YRP Hanger

**SILT**

FIGURE No.	
REF. No.	2005133
DATE	July 2024

## **ENCLOSURE 2**

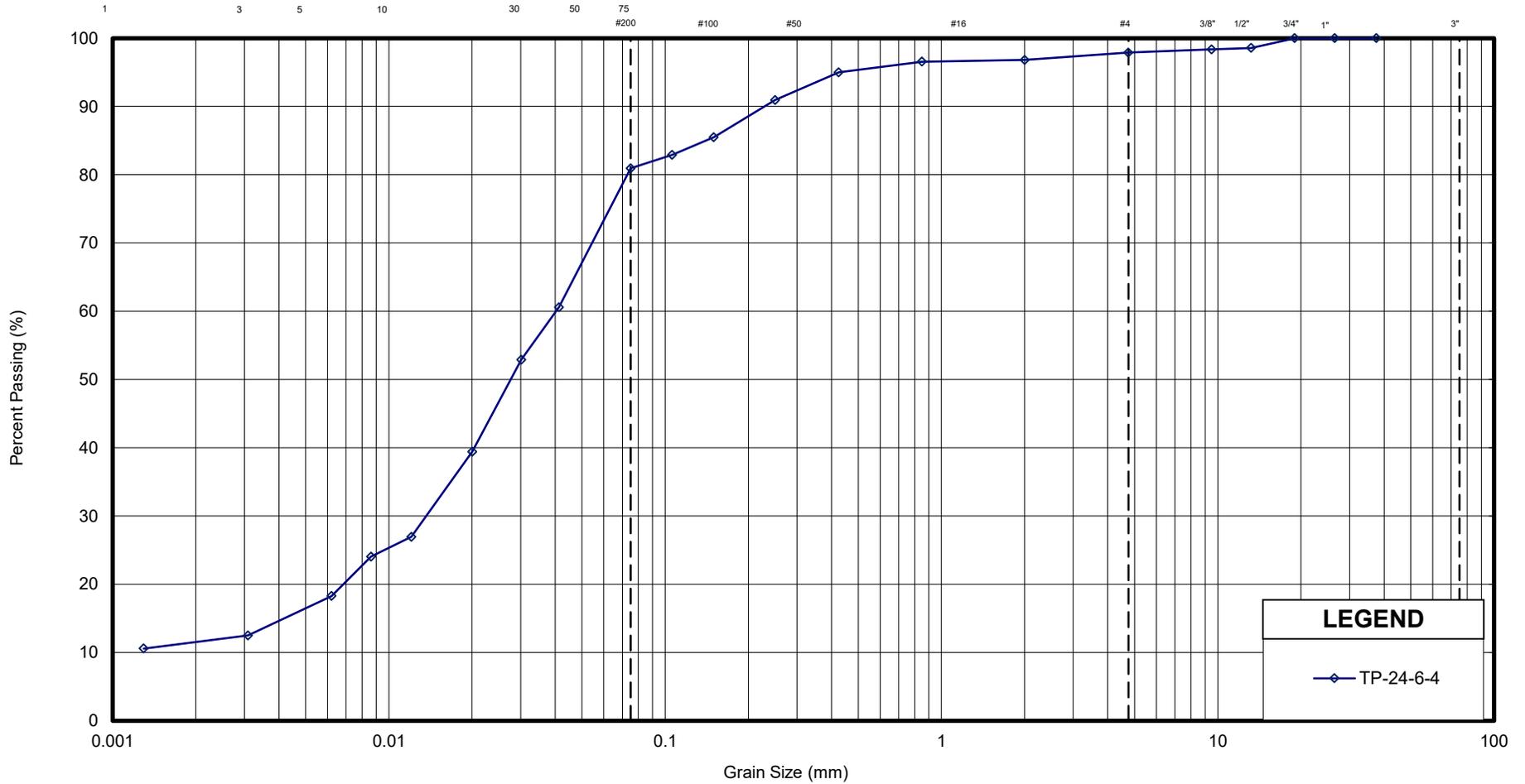
Grain Size Analysis (T-Time)

**UNIFIED SOIL CLASSIFICATION SYSTEM**

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (IMPERIAL)



LEGEND	
—◇—	TP-24-6-4

GEI Lab No.	Description	Gr.	Sa.	Si.	Cl.	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>
7837	SILT, Some Sand, Some Clay, Trace Gravel	2	17	69	12	-	0.014	0.040	-	-

	GRAIN SIZE DISTRIBUTION - Azimuth Environmental - YRP Hanger	FIGURE No.	
	<b>SILT</b>	REF. No.	2005133
		DATE	July 2024

## **ENCLOSURE 3**

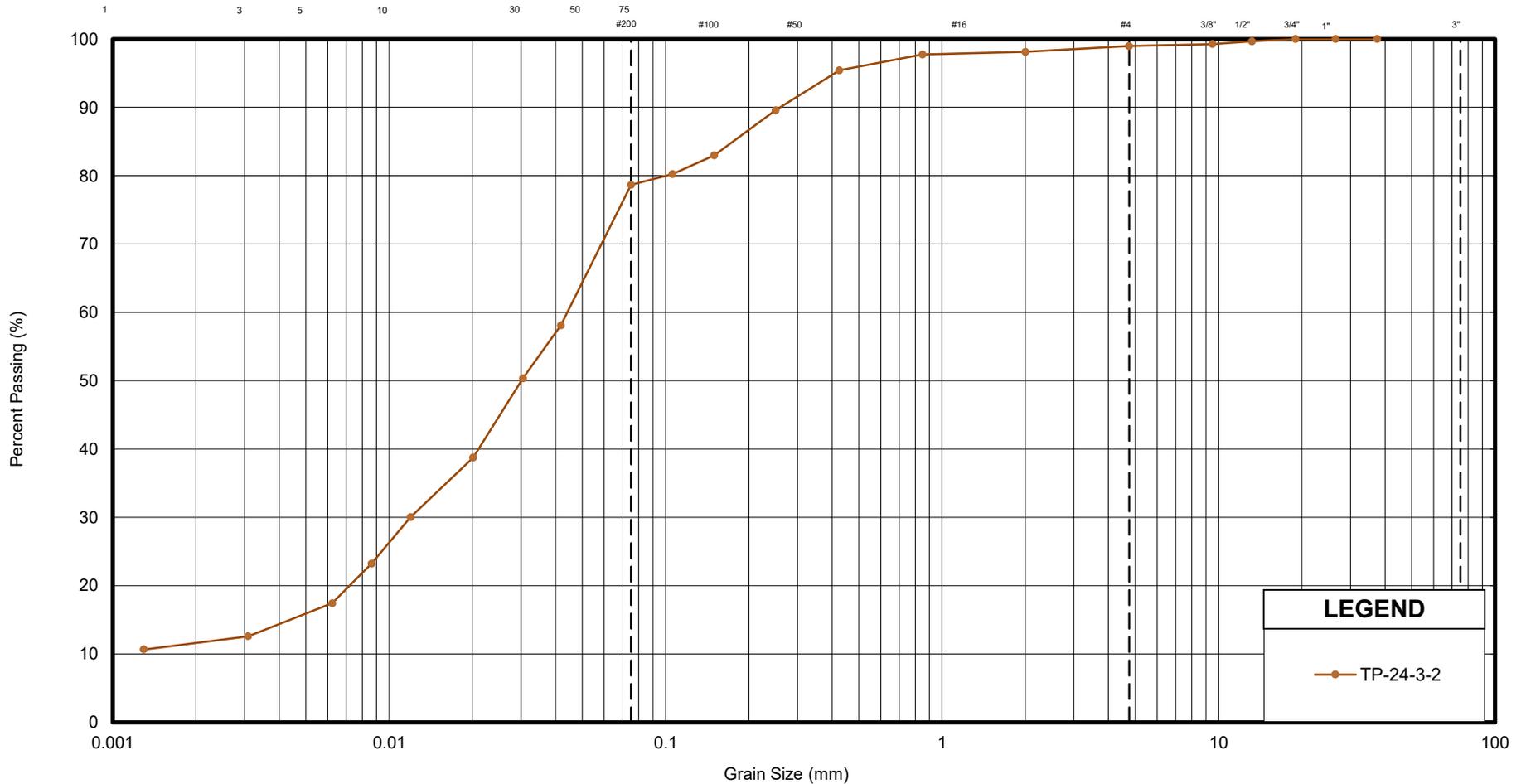
Grain Size Analysis (T-Time)

**UNIFIED SOIL CLASSIFICATION SYSTEM**

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

SIEVE DESIGNATION (IMPERIAL)



LEGEND	
—●—	TP-24-3-2

GEI Lab No.	Description	Gr.	Sa.	Si.	Cl.	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>
7838	SANDY SILT, Some Clay, Trace Gravel	1	20	67	12	-	0.012	0.044	-	-



GRAIN SIZE DISTRIBUTION - Azimuth Environmental - YRP Hanger

**SANDY SILT**

FIGURE No.	
REF. No.	2005133
DATE	July 2024

# TEST PIT LOG

## Environmental Assessments & Approvals

<b>Project Name/ Project Client</b>	YRP Hanger/ York Regional Police	<b>Project Address</b>	90 Bales Drive East, Sharon, ON	<b>Date</b>	July 23, 2024
<b>Test Pit Number</b>	TP24-1	<b>Contractor</b>	Provided by Proponent	<b>Elevation</b>	NA
<b>Operator / Equipment</b>	Brock Excavation / Track Mounted Excavator	<b>Test Pit Size</b>	1m x 3m	<b>Datum</b>	Ground Surface
<b>Temperature</b>	25°C	<b>Weather</b>	Sunny	<b>Sample Type</b>	Soil

Depth		Soil description	Samples		pH	Remarks / Chemical Analysis
From (m)	To (m)		No.	Depth (mbgs)		
0.00	0.30	Brown, dry, loose sandy topsoil with organics and rootlets.	1	-	-	-
0.30	2.10	<i>Fill:</i> Light brown, dry, compact silt w/ some fine sand and clay. Some mottling after 50 cm. Becoming moist at 70 cm.	2	-	-	Sample submitted for grain size and T-time assessment.
		<b>Test Pit Terminated at 2.1 mbgs</b>				
<b>Comments</b>			<b>Water Conditions in Test Pit</b>			
Standpipe not installed in test pit prior to backfilling.			<input type="checkbox"/> Wet upon completion <input checked="" type="checkbox"/> Dry upon completion			

**JOB No.** 24-054  
**TEST PIT No.** TP24-1  
**FIELD STAFF** B.Petterson



## TEST PIT LOG

### Environmental Assessments & Approvals

<b>Project Name/ Project Client</b>	YRP Hanger/ York Regional Police	<b>Project Address</b>	90 Bales Drive East, Sharon, ON	<b>Date</b>	July 23, 2024
<b>Test Pit Number</b>	TP24-3	<b>Contractor</b>	Provided by Proponent	<b>Elevation</b>	NA
<b>Operator / Equipment</b>	Brock Excavation / Track Mounted Excavator	<b>Test Pit Size</b>	1m x 3m	<b>Datum</b>	Ground Surface
<b>Temperature</b>	25°C	<b>Weather</b>	Sunny	<b>Sample Type</b>	Soil

Depth		Soil description	Samples		pH	Remarks / Chemical Analysis
From (m)	To (m)		No.	Depth (mbgs)		
0.00	0.20	Brown, dry, loose sandy topsoil with organics and rootlets.	1	-	-	-
0.20	0.40	<i>Fill:</i> Light brown, dry, compact silt w/ fine sand and some stone and clay.	2	-	-	Sample submitted for grain size and T-time assessment.
0.40	0.50	<i>Buried Organics :</i> Dark brown to black, lots of organic material and woody debris.	3	-	-	-
0.50	1.95	<i>Fill:</i> Light brown, dry, compact to dense silt w/ some fine sand and clay. Some mottling after 50 cm. Becoming moist at 1.1 cm.	4	-	-	-
		<b>Test Pit Terminated at 1.95 mbgs</b>				
<b>Comments</b>			<b>Water Conditions in Test Pit</b>			
Standpipe not installed in test pit prior to backfilling.			<input type="checkbox"/> Wet upon completion <input checked="" type="checkbox"/> Dry upon completion			

**JOB No.** 24-054  
**TEST PIT No.** TP24-3  
**FIELD STAFF** B.Petterson

## TEST PIT LOG

### Environmental Assessments & Approvals

<b>Project Name/ Project Client</b>	YRP Hanger/ York Regional Police	<b>Project Address</b>	90 Bales Drive East, Sharon, ON	<b>Date</b>	July 23, 2024
<b>Test Pit Number</b>	TP24-4	<b>Contractor</b>	Provided by Proponent	<b>Elevation</b>	NA
<b>Operator / Equipment</b>	Brock Excavation / Track Mounted Excavator	<b>Test Pit Size</b>	1m x 3m	<b>Datum</b>	Ground Surface
<b>Temperature</b>	25°C	<b>Weather</b>	Sunny	<b>Sample Type</b>	Soil

Depth		Soil description	Samples		pH	Remarks / Chemical Analysis
From (m)	To (m)		No.	Depth (mbgs)		
0.00	0.20	Brown, dry, loose sandy topsoil with organics and rootlets.	1	-	-	-
0.20	0.80	<i>Fill:</i> Light brown, dry, compact silt w/ fine sand and some stone and clay.	2	-	-	-
0.80	1.40	<i>Fill:</i> Dark grey, moist silty clay w/ some organics and trace sand. Refuse present (i.e., wood debris, concrete, wire, plastic, etc.).	3	-	-	-
1.40	2.30	<i>Fill:</i> Light brown, dry, compact silt w/ some fine sand and clay; trace organics. Refuse present (i.e., wood, concrete, plastic, etc.). Pocket of medium-coarse sand at 45 cm.	4	-	-	-
		<b>Test Pit Terminated at 2.1 mbgs</b>				
<b>Comments</b>			<b>Water Conditions in Test Pit</b>			
Standpipe not installed in test pit prior to backfilling.			<input type="checkbox"/> Wet upon completion <input checked="" type="checkbox"/> Dry upon completion			

**JOB No.** 24-054  
**TEST PIT No.** TP24-4  
**FIELD STAFF** B.Petterson





# YRP Hanger - Servicing Assessment

Test Pit Location Plan

## Legend

■ Test Pit Location



Google Earth

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100 m