

D.04 Surface Water Existing Conditions and
Impact Assessment Report; completed by
Dillon Consulting Limited and
Golder Associates Ltd., respectively

REPORT

Surface Water Existing Conditions and Impact Assessment

W12A Landfill Expansion EA, City of London, Ontario

Submitted to:

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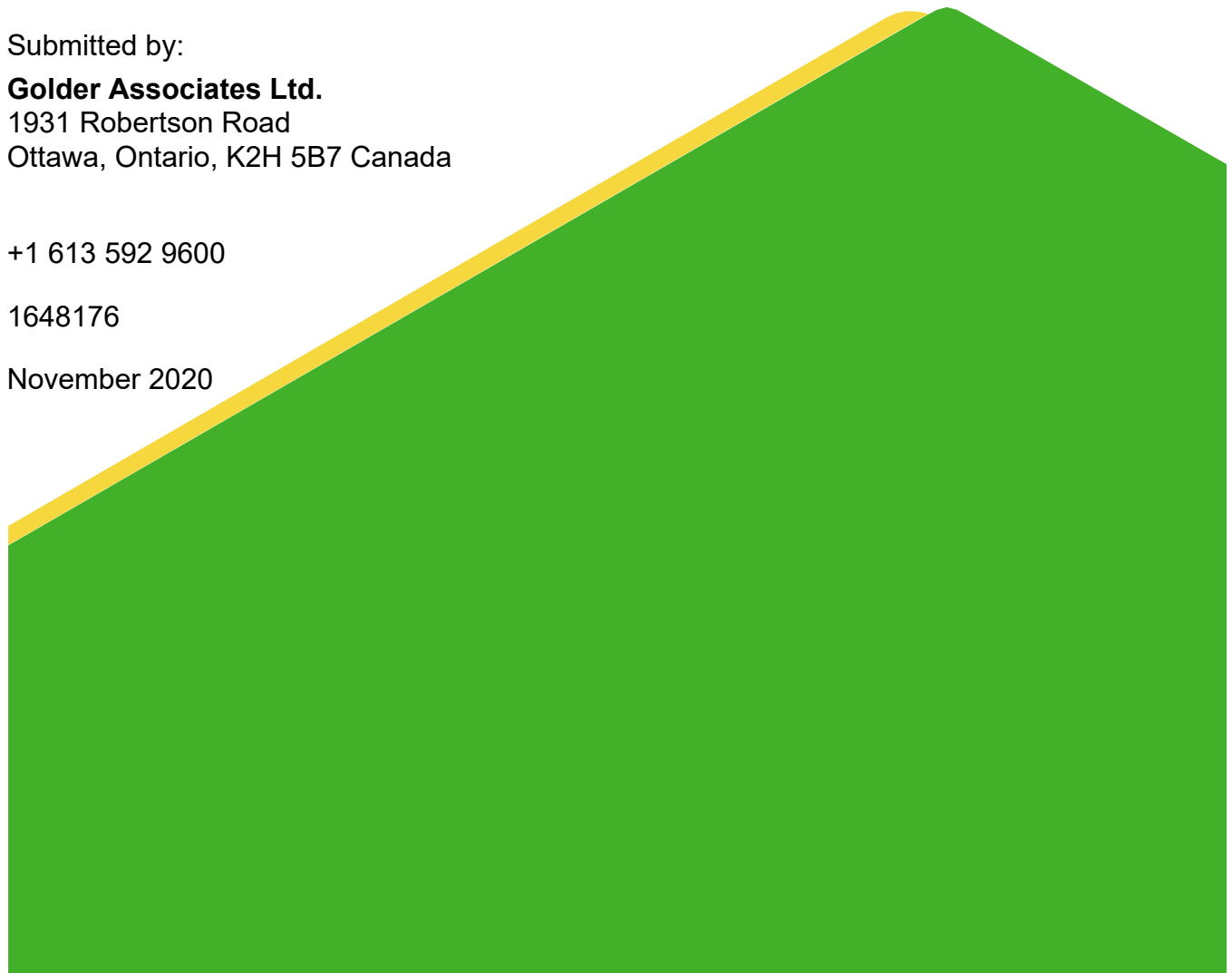
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ATTACHMENT A

Surface Water Existing Conditions Assessment – Dillon Consulting Limited

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Stormwater Management Existing Conditions and Impact Assessment – Golder Associates Ltd.

Acronyms and Glossary of Terms

Term / Acronym	Definition
EA	Environmental Assessment
EASR	Environmental Assessment Study Report
ECA	Environmental Compliance Approval
EPA	<i>Environmental Protection Act</i>
m	metres
m ³	cubic metres
masl	Metres Above Sea Level
MECP	Ministry of Environment, Conservation and Parks
MRF	Material Recovery Facility
OWRA	<i>Ontario Water Resource Act</i>
PWQO	Provincial Water Quality Objectives
SWM	Stormwater Management
ToR	Terms of Reference

1.0 INTRODUCTION

The City of London is proposing an expansion of the existing W12A Landfill site (the Project) located at 3502 Manning Drive, part of Lots 18, 19 and 20 of Concession 6 in the former Township of Westminster, in the south end of the City of London Ontario.

The environment is considered in broad terms to include the natural, social, cultural, and economic aspects of the environment. This document presents the details of the existing conditions and impact assessment related to surface water for the Environment Assessment (EA) of the proposed W12A Landfill expansion. This work has been completed in two parts; Dillon Consulting Limited (Dillon) completed the surface water existing conditions in Attachment A and Golder Associates Ltd. (Golder) completed the stormwater management existing conditions and impact assessment in Attachment B.

All work was conducted in accordance with the requirements set out in the approved Amended Terms of Reference (ToR) dated February 2019 and the surface water work plan dated August 2019.

This report is a supporting document to the Environmental Assessment Study Report (EASR).

2.0 PROJECT DESCRIPTION

The following describes the Project components considered relevant for the purposes of this surface water assessment.

The landfill expansion consists of a vertical expansion over the existing landfill footprint to provide 13.8 million cubic metres (m³) of airspace for waste and daily cover. To accommodate ancillary features and provide additional on-Site buffer width compared to the existing landfill, the northern boundary of the landfill property will be adjacent to Scotland Drive and a 100 metre (m) buffer width will be provided on the east side of the landfill footprint and some additional area in the southeast corner to tie into the adjacent Material Recovery Facility (MRF) property line. This results in a 106.3 hectare (ha) landfill footprint within a 192.4 ha landfill property, with a highest ridge elevation of waste of 318.43 metres above sea level (masl).

Additional details of the project pertinent to the surface water impact assessment are described in Attachment B, Section 2.1.

3.0 EXISTING CONDITIONS

The details and discussion of existing surface water conditions is provided in Attachment A by Dillon. Dillon first reviewed background reports related to surface water and surface water management on the W12A Landfill site and previous studies of the subwatersheds within which the W12A Landfill site is located, and then completed a site investigation on May 20, 2020 to confirm overland flow routes, drainage boundaries and outlet locations of surface water features. Dillon then prepared a summary of the existing surface water conditions.

4.0 IMPACT ASSESSMENT

A technical assessment of the existing stormwater management system on the W12A Landfill site and the assessment of impacts on the stormwater management system due to the proposed landfill expansion was completed by Golder. The completion of the impact assessment is provided in Attachment B. Golder prepared a technical assessment of the design of the existing stormwater management system (SWM ponds) including on-site drainage areas, storm storage volumes, design for suspended solids removal and outlet structure design to control the release of runoff water impounded from storm events. An assessment of the effects of changes in on-site drainage areas to each of the existing SWM ponds from the proposed expansion was then completed. The required changes to the SWM ponds to provide an enhanced level (80%) suspended solids removal for all site runoff, required increases in storage volumes to accommodate the range of storm events required by Ontario Regulation 232/98 as well as potential climate change (the 250 year return period storm event) and modifications to the SWM pond outlet structure design to maintain post-expansion flows to at or below pre-development flows were then determined.

An impact assessment of the potential effects of the expansion on surface water quantity and quality was prepared. This analysis indicates: 1) the changes in on-site drainage areas associated with the expansion is expected to have negligible effect on runoff and drainage within areas of the two subwatersheds downstream of the W12A Landfill site, and; 2) the modifications to the SWM system ponds and ditches will result in surface water quality conditions that are comparable to existing conditions and meet MECP Provincial Water Quality Objective (PWQO) requirements and prevent significant outflows that could potentially impair the water quality in downstream watercourses in extreme events.

The impact assessment concluded that no mitigation measures in addition to those proposed to modify the stormwater management system components are required.

5.0 SURFACE WATER AND STORMWATER MANAGEMENT MONITORING PROGRAMS

The City of London has an existing comprehensive surface water monitoring program for the existing landfill. Since the points of discharge from the stormwater management system (ponds) for the proposed expansion are the same as the existing site, it is proposed that the existing surface water monitoring program outlined in the current *Ontario Water Resources Act* (OWRA) Environmental Compliance Approval (ECA) be continued for the proposed expansion. The proposed surface water monitoring program for the landfill expansion is provided below in Tables 1 and 2.

The parameters to be tested for in the surface water are the same as the parameters suggested in the Landfill Standards (MECP, 1998).

The surface water sampling locations are shown on Figure 1 in Attachment A by Dillon Consulting.

Table 1: Surface Water Monitoring Program

Location	Parameters	Sampling Dates
Surface Water Sampling Stations Stations 1, 2, 3, 5, 7 and 10	Comprehensive List Indicator List	One spring and one fall sampling event Two other sampling events
Pond Outfall Pond 1, Pond 2/3, Pond 4 and Pond 5	Comprehensive List Indicator List	One spring and one fall sampling event Two other sampling events

Table 2: Surface Water Monitoring Parameters

Parameter	Surface Water		Parameter	Surface Water	
	Comp.	Indicator		Comp.	Indicator
General			General - Nutrients		
Alkalinity	X	X	Ammonia	X	X
Biological Oxygen Demand ₅	X	X	Nitrate	X	X
Chemical Oxygen Demand	X	X	Nitrite	X	X
Conductivity	X	X	Total Kjeldahl Nitrogen	X	X
Hardness	X	X	Total Phosphorus	X	X
pH	X	X	Trace Metals		
Phenols	X	X	Arsenic	X	
Sulphate	X	X	Barium	X	
Suspended Solids	X	X	Boron	X	
Total Dissolved Solids	X	X	Cadmium	X	
Alkalinity	X	X	Chromium	X	
Major Ions			Copper	X	
Chloride	X	X	Lead	X	
Iron	X	X	Mercury	X	
Field			Zinc	X	
Conductivity	X	X			
Dissolved Oxygen	X	X			
Flow ^(a)	X	X			
pH	X	X			
Temperature	X	X			

Notes:

(a) Flows conditions (i.e., stagnation, qualitative description of flow rate, etc.) in water discharged from each stormwater management pond will be noted.

6.0 OTHER APPROVALS

The stormwater management aspects of this EA will also require approval under Section 52 of the OWRA in the form of an amended ECA. The overall surface water component of the expansion will require approval under the *Environmental Protection Act* (EPA) as part of an amended ECA for waste disposal.

ATTACHMENT A

**Surface Water Existing Conditions Assessment –
Dillon Consulting Limited**

Memo



To: Mike Losee, City of London (City)
From: Debra Ballard, Dillon Consulting Limited (Dillon)
cc: Paul Donahue, Dillon
Rob Kell, Dillon
Wesley Abbott, City
Date: July 15, 2020
Subject: Surface Water Existing Conditions Assessment W12A Landfill
Our File: 17-6665

The following information provides a review of background information available to take into consideration for the Alternative Methods assessment for the proposed expansion of the existing W12A Landfill.

1. Background Studies

1.1. Earth Tech Stormwater Management Master Plan (2002)

The W12A Stormwater Master Plan was undertaken to review the existing stormwater management plan and identify a preferred stormwater management plan for the Site. The Stormwater Master Plan aimed to ensure that drainage leaving the Site does not adversely affect surface water resources downstream, and that internal stormwater drainage does not adversely affect site operations.

Specific stormwater objectives for the subject site were detailed as follows:

- Control flows from the site so there is no increase in peak flow in the downstream receiving watercourses, and no increase in flooding, for design flow events up to a 100-year storm
- Over-control peak flows from more frequent storms (i.e., the 5-year and less severe events) to reduce the potential for erosion in the downstream watercourses
- Minimize the sediment loading into the downstream watercourses by reducing the generation of sediment and providing for the removal of suspended sediment from site stormwater prior to off-site discharge
- Provide a significant level of enhancement to the downstream habitat potential by releasing stormwater over extended periods of time, and enhancing baseflow in the watercourses where feasible
- Control runoff from active landfill areas to limit the potential for off-site pollutant migration.

The report describes the stormwater management design parameters proposed in the Master Plan. Of note to the continuing development of the landfill are the following:

- Post development impervious level for the landfill will be 55%, based on modelling results. The runoff is generally similar to that expected for an urban area with 55% impervious area
- Level 2 control was used for determining the water quality storage requirements for catchments within the Dodd Creek Subwatershed (70 m³/ha permanent volume, 40 m³/ha active volume)
- Level 1 control was used for determining the water quality storage requirements for catchments within the Dingman Creek Subwatershed (150 m³/ha permanent volume, 40 m³/ha active volume)
- For the Dodd Creek Subwatershed, the settling of sediment is facilitated by long detention times and slow release rates for all storms less intense than the 5-year design event
- Proposed erosion control storage will be at least as large as the requirement identified in the main body of the report of the Dingman Creek Subwatershed study (375 m³/ha).

It is noted that Pond 1 is the only pond with an active storage volume that meets the erosion control requirements detailed in the Stormwater Master Plan, with the other three ponds being undersized. This does not take into consideration the requirement for attenuation of rainfall events up to the 100-year event, the storage volume for which would be in addition to the erosion control volume.

1.2. Dillon Consulting Limited Surface Water Background Study (2005)

This report was undertaken in 2005 to assist in the planning process and continued evolution of the W12A Landfill facility into an integrated waste management facility. The investigation included a background area study, surface water assessment, geomorphic analysis and the development of a surface water management plan.

This report describes the stormwater management design parameters used in the plan. Of note to the continuing development of the landfill are the following:

- Existing stormwater ponds within the W12A Landfill site were being upgraded to provide the highest level of quality control of stormwater runoff and further reduce Total Suspended Solids load to downstream receiving watercourses
- Many streams within the Study Area have been altered to improve drainage characteristics by straightening and deepening the channels. Deep, confined channels tend to exhibit substantially higher peak flow velocities (due to small top-width and deep flow depth) compared to unconfined channels with equivalent flow rate. Natural stream morphology and associated instream habitat have been significantly impacted in the altered streams
- Very few (less than 10%) of the watercourses within the study's study area have vegetated streambanks, typically consisting of woody species and, as a result, banks are unstable, erosion and scour is prevalent, water

temperatures are high and soil transported from adjacent lands readily enters the watercourses

- Thirteen pipe culverts and one rigid frame open footing box culvert exist around the perimeter of the existing landfill area.

2. Subwatershed Studies

The existing W12A Landfill is situated between two large subwatersheds. Divided diagonally from the southwest to northeast corner, the northerly portion of the Site falls within the Dingman Creek Subwatershed (approximately 13 ha), while the southerly portion of the Site Study Area falls within the Dodd Creek Subwatershed (remainder of Site). The Dingman Creek subwatershed falls within the jurisdiction of the Upper Thames River Conservation Authority while Dodd Creek falls under the regulatory powers of the Kettle Creek Conservation Authority.

2.1. Dingman Creek Subwatershed Background Study

The original Dingman Creek Subwatershed Study (DCSS) was initiated in October 1992 by the City of London. In 2005, the DCSS was updated (2005 DCSSU). The update was undertaken by Delcan Corporation under the direction of the City of London with the direct involvement and guidance of a Technical Advisory Committee. In February 2016, Parsons Corporation updated the 2005 DCSSU (2016 DCSSU), and this updated document was used for this assessment.

The 2005 DCSSU included a stormwater management strategy for the subcatchment within which the landfill site is located (Tributary R). This included detaining the 25 mm 24 hour storm in new urban areas and releasing it over a period of up to 72 hours for erosion control, and control of post development flow to predevelopment flow for the 1:100 year rainfall event.

The 2016 DCSSU report suggests that peak attenuation control measures from the 2005 DCSSU are being met in the majority of cases, and erosion control measures are effectively meeting the erosion target. The 2016 DCSSU does not specify new or different standards to be met moving forward.

2.2. Kettle and Dodd Creek Subwatershed Study

The Subwatershed Plan for the Upper Kettle Creek and Dodd Creek Subwatersheds was completed in May 1995. This plan provided a series of management options that could be applied together to meet the objectives of the plan. It should be noted that at the time this report was completed, the subwatershed did not have future growth proposed, and the recommended plan addressed problems resulting from existing land uses.

The plan indicates that flooding of buildings and other structures within the Kettle and Dodd Creek subwatersheds is not known to be a problem under existing conditions and no strategy was proposed for reducing flooding within the

catchment area. It also noted that detailed floodplain mapping should be prepared on an as needed basis along those watercourses where urban/rural development may occur.

The plan also detailed that existing water quality within the subwatershed was generally poorer than other subwatersheds under study in the City of London, including high wet weather concentrations of suspended solids. Quality/quantity control measures suggested for rural areas included the options of installing settling basins for stormwater runoff.

No recommendations for erosion control via detention and slow release were provided within the plan.

3. Current Existing Conditions within the Landfill Site

Information on the existing approved stormwater management facilities installed at the Site Study Area were obtained from the Amended Certificate of Approval Municipal and Private Sewage Works Number 4175-8C4SD5 (2011). It provided catchment areas, permanent and active storage volumes and outlet structure information for each of the four ponds present on the landfill site. This information is summarized in **Table 1** below.

Table 1: Existing Pond Information Summary

Pond Name	Catchment Area (ha)	Permanent Storage Volume (m³)	Active Storage Volume (m³)	Outlet Structure Details
Pond 1	14.4	2470 (at 273.2 masl)	5800	150 mm diameter Hickenbottom outlet structure inside a 1500 mm diameter perforated corrugated steel pipe (CSP) riser structure with clear stone filter discharging through a 1200 mm diameter manhole equipped with 150 mm diameter gate valve into an existing 900 mm diameter CSP culvert to Drainage Ditch 7.
Pond 2/3	29.6	2660 (at 271.2 masl)	11100	300 mm diameter Hickenbottom outlet structure placed inside a 1500 mm diameter perforated CSP riser structure connected to a 29 m long 300 mm diameter polyvinyl chloride (PVC) pipe passing through a 1200 mm diameter manhole equipped with 300 mm

				diameter gate valve discharging into an existing outlet ditch and one ditch inlet structure with invert elevation of 272.2 metres above sea level (masl) discharging through a 18 m long 750 mm diameter concrete pipe into an existing outlet Drainage Ditch 5.
Pond 4	57.7	5200 (at 269.7 masl)	20200	125 mm diameter Hickenbottom outlet structure placed inside a 1500 mm diameter perforated CSP riser structure with clear stone filter, discharging through a 29 m long 150 mm diameter non-perforated PVC pipe equipped with a 150 mm gate valve into a 1200 mm diameter manhole and then through a 29 m long 450 mm diameter pipe into an outlet ditch equipped with a 1650 mm x 900 mm concrete box with open grated top at invert elevation of 271.0 masl and discharging through an existing 1050 mm diameter pipe into Drainage Ditch 3.
Pond 5	41.0	3690 (at 268.5 masl)	11800	900 mm diameter pipe into an outlet structure manhole equipped with an overflow weir at elevation 269.4 masl, a 600 mm x 600 mm slide gate valve with a 150 mm diameter orifice, discharging through 900 mm diameter outlet pipe into Drainage Ditch 1.

The information provided in this document differed from the original design volumes for the stormwater management facility in the Stormwater Master Plan, and likely reflects the upgrading of these devices as was described in the Surface Water Background Study (2005). The permanent storage volume provided in SWM Pond 1 is sufficient to meet Enhanced 80% long-term suspended solids removal, all other ponds have permanent storage volumes sufficient to provide *Normal* 70% long-term suspended solids removal.

3.1. Existing Environmental Conditions

A site investigation was undertaken on May 20, 2020, by Dillon Consulting staff to confirm overland flow routes, drainage boundaries and outlet locations of surface water features. The results of this field investigation are summarised in the following subsections.

3.1.1. Inventory of Existing Hydraulic Structures and Surrounding Characteristics

The following section provides an overview of the existing surface water features adjacent to, or receiving flow from the W12A Landfill as surveyed on May 20, 2020. There were 14 hydraulic structure locations surveyed. They consist of pipe culverts or tile drains of various diameters and one rigid frame open footing box culvert. The location of the surface water structures are shown on Figure 1, attached; the location numbers correspond to a condition assessment, brief description and photo within the text of this section.



Photo 1: Location 1 – Culvert Inlet



Photo 2: Location 1 – Culvert Outlet

Location 1, shown in Photo 1 and Photo 2 consists of a 250 mm diameter CSP culvert that directs ditch flow from the south side of Scotland Drive to the north. At the inlet of the culvert there is a narrow defined channel with some standing water, but no flow through the culvert at the time of survey. The base of the culvert has structurally deteriorated at both the upstream and downstream ends. At the downstream end of the culvert there has been erosion of bed material from under the culvert outlet and the surrounding bank. The downstream end of the culvert is moderately perched with the pipe outlet approximately 200 mm above the channel invert. At the time of survey there was standing water in the pool under the culvert and some debris in the ditch drain around the culvert.



Photo 3: Location 2 – Culvert Inlet



Photo 5: Location 2 – Downstream Channel



Photo 4: Location 2 – Culvert Outlet

Location 2, shown in Photo 3, Photo 4 and Photo 5 consists of a 375 mm diameter CSP culvert that directs ditch flow from the south side of Scotland Drive to the north. At the upstream end of the culvert, defined channels 300-400 mm wide convey flow from the east and west along Scotland Drive. The upstream inlet to the culvert was damaged and partially submerged. There is standing water at the inlet to this culvert, but it was not flowing at the time of the survey.

The culvert discharges into a well-defined ravine that meanders through the woodlot to the north. The ravine in which the watercourse flows exhibits signs of significant erosion and mass wasting; attempts to stabilize the road embankment have been done through the use of broken concrete and rip-rap (as noted in photos). At the time of survey there was no water flowing through the culvert pipe; however, there was moving water within the watercourse downstream of the culvert outlet erosion protection.



Photo 6: Location 3 – Culvert Inlet



Photo 8: Location 3 – Culvert Outlet



Photo 7: Location 3 – Downstream Channel

Location 3 consists of a 300 mm diameter CSP culvert, which directs ditch flow from the south side of Scotland Drive to the north. The inlet to this culvert is partially buried with sediment deposition to approximately half of the culvert barrel height. At the time of survey there was less than 1 L/s flowing into this culvert, it is unclear where this water was discharging. The outlet of the culvert is covered by grass and there was no outflow from the culvert at the time of survey.

Downstream of the culvert, there is a defined channel with silt deposition along the base to a grated ditch inlet to a closed tile drain. The outlet of this closed system was not able to be determined at the time of survey and is not apparent from aerial photographs.



Photo 9: Location 4 – Culvert Inlet



Photo 10: Location 4 – Culvert Outlet

Location 4, shown in Photo 9 and Photo 10, consists of an unknown diameter culvert, which directs ditch flow from the south side of Scotland Drive to the north. The upstream inlet to this culvert was completely submerged by ponded water and deposited sediment at the time of survey. The outlet to this culvert was not able to be found at the time of survey as it was covered with vegetation. However, there were indicators that water had flowed through the area.



**Photo 11: Location 5 –
Upstream Pond**



**Photo 13: Location 5 – Culvert
Outlet**



**Photo 12: Location 5 – Culvert
Inlet**

Location 5, shown in Photo 11, Photo 12 and Photo 13 consists of a 525 mm diameter CSP pipe that directs ditch flow from the south side of Scotland Drive to the north. Approximately 100 m upstream of the pipe is a watering hole/pond area (Pond A) with no defined outlet that may retain some flow from the catchment. The culvert inlet is partially submerged with 100 mm of standing water at the inlet.

Approximately 3 m downstream of the culvert inlet a joint between pipe sections has broken cleanly, such that the inlet section of pipe is completely detached from the remainder of the culvert. An empty steel drum is lying across the culvert near the inlet, and another further upstream near the ponded area (as visible in **Photo 11**). Subsequent to the site visit, the City of London removed the drums from the area.

At the downstream outlet there is indications of erosion within the roadside channel, with vegetation roots and utility cable exposed. The culvert outlets to a pool of standing water, approximately 3 m by 5 m surface dimension with a ponded depth of 450 mm. The base of the pipe outlet has deteriorated.



Photo 14: Location 6 – Culvert Inlet



Photo 15: Location 6 – Culvert Outlet

Location 6, shown in Photo 14 and Photo 15, consists of a 300 mm diameter CSP culvert that directs ditch flow from the south side of Scotland Drive to the north. The inlet to this culvert is partially buried with sediment disposition to approximately half of the culvert barrel height. At the time of survey there was less than 1 L/s flowing through this culvert. The outlet from this culvert is perched, with erosion of the road embankment beneath the pipe and behind the outlet. This erosion has undercut the vegetation above the pipe and exposed the roots of nearby trees. The undercutting of the outlet has led to the destabilization of the pipe outlet and a section of pipe has broken from the culvert and is lying at the base of the eroded bank. North of Scotland Drive the watercourse continues in a northerly direction through the wooded area.



Photo 16: Location 7 – Culvert Inlet



Photo 17: Location 7 – Culvert Outlet

Location 7, shown in Photo 16 and Photo 17 consists of a 600 mm diameter concrete pipe culvert that directs flow from the drainage area on the south side of Scotland Drive to the north. At the culvert inlet, rip rap has been used to stabilise the bank and channel upstream of the culvert, water depth in the culvert was 150 mm at the upstream end. Depth of flow at the outlet was 200 mm at the time of survey. At the time of survey, the culvert was flowing at less than

1 L/s. The culvert discharges into a grassy channel approximately 450 mm wide with a flow depth of 160 mm at the time of survey. Rip rap has been used to stabilise the bank above the culvert outlet.

The watercourse forms part of a system of ponds, which originates in an undulating meadow with mixed land use to the south of Scotland Road. The eastern pond (Pond C) does not have an outlet structure. Instead, during rainfall, water in excess of the pond capacity will overflow in a westerly direction along the roadside ditch toward the western pond (Pond B). At the time of survey, a small earthen bund had been constructed to raise the water level in the pond. This bund would overtop and likely wash away in a major rainfall event. The outlet from the western pond is controlled by a 500 mm diameter outlet pipe, which conveys water 100 m to an outlet near the Location 7 culvert. Along this outlet pipe a 600 mm x 600 mm grate has been installed in the roadside ditch.



Photo 18: Location 8 – Box Culvert Upstream



Photo 19: Location 8 – Box Culvert Downstream



Photo 20: Location 8 – Outlet Upstream of Box Culvert



Photo 21: Location 8 – Outlet Upstream of Box Culvert

Location 8, shown in Photo 18 to Photo 21 consists of a 3.75 m wide concrete frame open footing box culvert, which directs flow from the north side of Manning Drive to the south. Upstream of the box culvert is a 450 mm diameter plastic

outlet within a concrete headwall, with a rodent grate (shown on Photo 20), and a 250 mm diameter CSP surrounded by rip rap. These pipe outlets were not present during the 2005 Dillon Surface Water Background Study. Upstream of the culvert is a well-defined channel flowing at approximately 30 L/s at the time of survey. There was some sediment deposition along the right bank of the watercourse within the culvert and across the width of the culvert at the downstream end. This is consistent with the agricultural land use within the upstream drainage areas.

The culvert discharges into a well-defined channel (Shore Drain). Upstream of Manning Drive the channel meanders through the agricultural lands within the Site-vicinity Study Area. Generally, the banks of the watercourse appear stable and are well vegetated with some undercutting. Bird nests were present along the upper corners of the culvert throughout its length at the time of the survey.



**Photo 22: Location 9 – Pond 5
Outlet**



**Photo 23: Location 9 – Pipe Outlet
Upstream of Culvert**



**Photo 24: Location 9 – Culvert
Inlet**



**Photo 25: Location 9 – Grate Inlet
near Culvert**



Photo 26: Location 9 – Culvert Outlet

Location 9, shown in Photo 22 to Photo 26 consists of a cluster of hydraulic structures at the southeast corner of the landfill site. Within the landfill property boundary is a 900 mm diameter CSP outlet pipe from Stormwater Management (SWM) Pond 5. Downstream of this pipe is a landfill flow monitoring station consisting of a pool of water with a concrete surround, and a v notch discharge weir.

Pooling on the upstream side of the flow monitoring station results in the Pond 5 outlet being submerged to approximately half the pipe height. This flow monitoring station was not recorded as part of the 2005 Dillon Surface Water Background Study. The concrete retaining structure is in poor condition, with part of the concrete wall falling into the ponded area and being held up with steel T-bars and a fallen tree partially blocking the outflow weir.

Downstream of the flow monitoring station is a 450 mm diameter acrylonitrile-butadiene-styrene (ABS) pipe in a concrete headwall discharging into the roadside ditch. The area around this outlet was covered with grass vegetation and no erosion was visible at the time of survey.

Also downstream of the flow monitoring station at Location 12 is an 800 mm diameter concrete culvert conveying water in a north to south direction under Manning Drive and a ditch inlet catch basin to a closed tile system. The roadside ditch in this area is clearly defined, with some rip rap erosion protection around the culvert inlet and a concrete headwall above the ditch inlet.

On the south side of Manning Drive the culvert outlet is surrounded by rip rap for erosion protection and immediately downstream of the culvert is a grated ditch inlet to a closed tile drain system.



Photo 27: Location 10 – Outlet Pipe from Landfill



Photo 28: Location 10 – Culvert Inlet



Photo 29: Location 10 – Culvert Outlet

Location 10, shown in Photo 27, Photo 28 and Photo 29 consists of a 1050 mm diameter CSP outlet pipe from the landfill area (Pond 4 outlet) and a 750 mm diameter plastic culvert that conveys water from north to south under Manning Drive. The CSP outlet pipe is bowed in the middle due to the weight of the soil embankment and the outlet has been deformed to 900 mm height. The outlet pipe discharges to a pool of standing water on the north side of Manning Drive with an approximately 3 m x 3 m surface area.

The standing water depth was approximately 350 mm, with approximately 150 mm of soft sediment deposited at the invert of the pipe. The outlet was not flowing at the time of survey.

The upstream end of the culvert under Manning Drive at Location 10 is submerged to approximately 450 mm depth in the pool of standing water described above. The southern/downstream end of this culvert is submerged in standing water. Some rip rap protection is visible around the edges of the pooled water. The pool and downstream channel were stable and vegetated and showed no signs of erosion at the time of survey.



Photo 30: Location 11 – Outlet from Landfill



Photo 31: Location 11 – Culvert Inlet



Photo 32: Location 11 – Culvert Outlet

Location 11, shown in Photo 30, Photo 31 and Photo 32 consists of an unknown diameter CSP outlet pipe from the landfill area and a 600 mm diameter CSP culvert, which conveys water from north to south under Manning Drive. The outlet pipe from the landfill was located between the two boundary fences of the landfill area and was not able to be accessed at the time of survey. The outlet was not flowing at the time of survey.

The inlet to the CSP culvert at this location is located in a grassy ditch drain on the north side of Manning Drive. There was localized standing water at the inlet to this culvert to a depth of 100 mm. The outlet to the culvert was covered with vegetation at the time of survey and was not able to be seen. The downstream channel was stable and vegetated and showed no signs of erosion at the time of survey.



**Photo 33: Location 12 –
Culvert Inlet**



**Photo 34: Location 12 – Culvert
Outlet**

Location 12, shown in Photo 33 and Photo 34 consists of unknown 1050 mm diameter CSP outlet pipe from the landfill area (Pond 2/3 outlet) and a 600 mm diameter CSP culvert that conveys water from north to south under Manning Drive. The outlet pipe from the landfill was located between the two boundary fences of the landfill area and was not able to be accessed at the time of survey. Flow from this outlet pipe was being conveyed to the downstream culvert via a defined channel 400 mm wide with a flow depth of 100 mm. At the time of survey this was flowing at less than 10 L/s.

The inlet to the CSP culvert at this location is located in a grassy ditch drain on the north side of Manning Drive. There was localized standing water at the inlet and outlet of this culvert at a depth of 100 mm. The outlet to the culvert was submerged and covered with vegetation at the time of survey. The downstream channel was stable and vegetated and showed no signs of erosion at the time of survey.



**Photo 35: Location 13 – Outlet
Pipe From Pond 1**



Photo 36: Location 13 – Culvert Inlet



**Photo 37: Location 13 –
Culvert Outlet**

Location 13, shown in Photo 35, Photo 36 and Photo 37, consists of an unknown (estimated 900 mm) diameter CSP outlet pipe from the landfill area and an 825 mm diameter CSP culvert which conveys water from east to west under White Oak Road. The outlet pipe from the landfill is the outlet from Pond 1. It was located between the two boundary fences of the landfill area and was not able to be accessed at the time of survey. Based on visual inspection from the road it was estimated to be 900 mm diameter.

Flow from this outlet pipe was being conveyed to the downstream culvert via an undefined channel thorough a grassed embankment. At the time of survey, this was flowing at less than 10 L/s.

The inlet to the CSP culvert at this location is located in a grassy ditch drain on the north side of Manning Drive. An attempt has been made to stabilize the headwall of this culvert by stacking broken concrete blocks. As a result, the inlet to the culvert was blocked approximately 1/3 of its open area by broken concrete at the time of survey. Standing water depth at the inlet and outlet of this culvert was 400 mm. The outlet to the culvert was squashed vertically. The downstream channel was stable and vegetated and showed no signs of erosion at the time of survey.



**Photo 38: Location 14 –
Culvert Inlet**



**Photo 39: Location 14 – Culvert
Outlet**

Location 14, shown in Photo 38 and Photo 39 consists of a 450 mm diameter CSP culvert, which directs ditch flow from the east side of White Oak Road to the west. The upstream end of the culvert is located in a grassed roadside ditch drain and has some rip rap stabilisation along the inlet pipe. The culvert was dry

at the time of survey. The culvert discharges into a well-defined channel that meanders through the farm field to the west. At the time of survey there was standing water approximately 100 mm deep at the outlet of the culvert, and the pipe was partially covered by vegetation.

3.1.2. Overland Flow Routes and Drainage Boundaries

In general, the local drainage boundaries observed on the Site Study Area were consistent with those detailed in the 2005 Dillon Surface Water Background Study Report. These boundaries have been refined based on the additional culverts observed under Scotland Drive during the field investigation. The revised drainage boundaries are shown on **Figure 1**.

3.1.3. Water Quality and Quantity Parameters

Currently, the W12A Landfill environmental monitoring program includes the monitoring of surface water quality in ditches and the surface water ponds. The results of the surface water monitoring completed in 2019 are summarized below.

Surface water samples were collected from six drainage ditches (referred to as Ditch 1, Ditch 2, Ditch 3, Ditch 5, Ditch 7 and Ditch 10; see description in **Table 2** below, which also references locations shown on **Figure 1**) and the four Stormwater Management Ponds (referred to as Ponds #1, #2/3, #4 and, #5).

Table 2: Surface Water Sampling Locations

Location Name	Location	Receiving Watershed	Completed Fill Area	Landfill Property	Comments
Ditch 1 (Location 9)	Weir at southeast corner of landfill	Dodd Creek	Yes	Yes	Receives flow from Pond #5
Ditch 2 (Immediately west of Location 9)	Weir at end of culvert at entrance to landfill	Dodd Creek	No	Yes	Includes drainage from buffer area including drop-off area
Ditch 3 (Location 10)	Weir, Manning Drive (centre of landfill)	Dodd Creek	Yes	Yes	Receives flow from Pond #4 and buffer area

Location Name	Location	Receiving Watershed	Completed Fill Area	Landfill Property	Comments
Ditch 5 (Location 12)	Ditch at southwest corner of landfill	Dodd Creek	Yes	Yes	Includes drainage from Pond #2/3 and buffer area
Ditch 7 (Location 13)	Ditch at northwest corner of landfill	Dingman Creek	No	Yes	Includes drainage from Pond #1 and agricultural land
Ditch 10 (Location 5)	North end of culvert at 3713 Scotland Drive	Dingman Creek	No	No	Includes drainage from agricultural land

Since the landfill is situated on a surface water divide, there are no sampling locations upstream from the landfill. Ditches #7 and #10 do not receive surface water from areas that have been landfilled and can be considered representative of background conditions.

For compliance purposes, three locations (Ditch 1, Ditch 3 and Ditch 5) and four water quality parameters (un-ionized ammonia, BOD₅, chloride and sulphate) are used as the trigger mechanism for mitigation measures. There were no exceedances for any trigger mechanism parameters in 2019 as shown in Table 3.

Table 3: Surface Water Quality – Compliance Locations

Parameter	Sampling Location	Results	
		Maximum (mg/L)	Average (mg/L)
BOD5 Trigger > 10 mg/L	Ditch #1	6	4
	Ditch #3	4	3
	Ditch #5	4	3
Chloride Trigger > 800 mg/L	Ditch #1	219	117
	Ditch #3	157	92
	Ditch #5	221	145
Sulphate Trigger > 1,000 mg/L	Ditch #1	38	18
	Ditch #3	99	59
	Ditch #5	201	121
Un-ionized Ammonia Trigger > 0.3 mg/L	Ditch #1	0.005	0.002
	Ditch #3	0.010	0.004
	Ditch #5	0.002	0.001

The City also undertakes sampling directly from the stormwater management ponds that receive runoff from areas that have been landfilled and covered (Ponds #2/3, #4 and #5). None of the samples collected from the stormwater management ponds exceeded any trigger mechanism parameters in 2019.

In most cases, average levels of the trigger mechanism parameters in ditches and ponds receiving water from the landfill were not significantly greater than average background concentrations in Ditches #7 and #10 in 2019. The exceptions included elevated sulphate concentrations in Ditch #5 and elevated un-ionized ammonia concentrations in Pond #4. The elevated levels of these parameters are fairly consistent with past results. Sulphate concentrations in Pond #2/3 were elevated compared to background conditions and historical results but had decreased compared to 2018 levels. In 2019, Ditch #2 had elevated chloride concentrations in two sampling events. Elevated chloride concentrations have occurred periodically over the past two decades at this location, which has been attributed to road salt impacts from areas frequently travelled by vehicles such as the landfill entrance and drop-off depot.

Other Parameters

Levels of suspended solids observed in 2019 were typically below 100 mg/L, which is similar to historic values. In 2019, the suspended solids levels in drainage ditches ranged from <3 mg/L to a maximum of 393 mg/L observed in Ditch #7 in October.

Phosphorus levels in all the ditches and the stormwater management ponds tested, including the ditches that do not receive surface water from the landfill, exceeded the Provincial Water Quality Objectives (PWQOs) criteria of 0.03 mg/L. In 2019, phosphorus levels generally ranged between 0.03 mg/L and 0.30 mg/L, which was below the discharge criterion of 0.40 mg/L in the City's Wastewater Discharge By-law.

In August 2019, at Ditches #1 and #2, phenol concentrations of 2.0 µg/L were measured, nominally exceeding the PWQO of 1 µg/L.

Metals levels were below the corresponding PWQOs for nearly all surface water samples. The PWQO for zinc (30 µg/L) was exceeded in the October 2019 sampling event at Ditch #3, which had a concentration of 251 µg/L. Historically, the stormwater management ponds had occasional exceedances of the PWQOs for copper and zinc.

Overall, there were no exceedance of trigger levels in the 2019 surface water monitoring program and surface water quality was comparable to historical surface water data.

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						W12A LANDFILL ALTERNATIVES ASSESSMENT	SHEET No.
						EXISTING SURFACE WATER FEATURES	FIG 1

ATTACHMENT B

**Stormwater Management Existing Conditions
and Impact Assessment –
Golder Associates Ltd.**

REPORT

Stormwater Management Existing Conditions and Impact Assessment

W12A Landfill Expansion EA, City of London, Ontario

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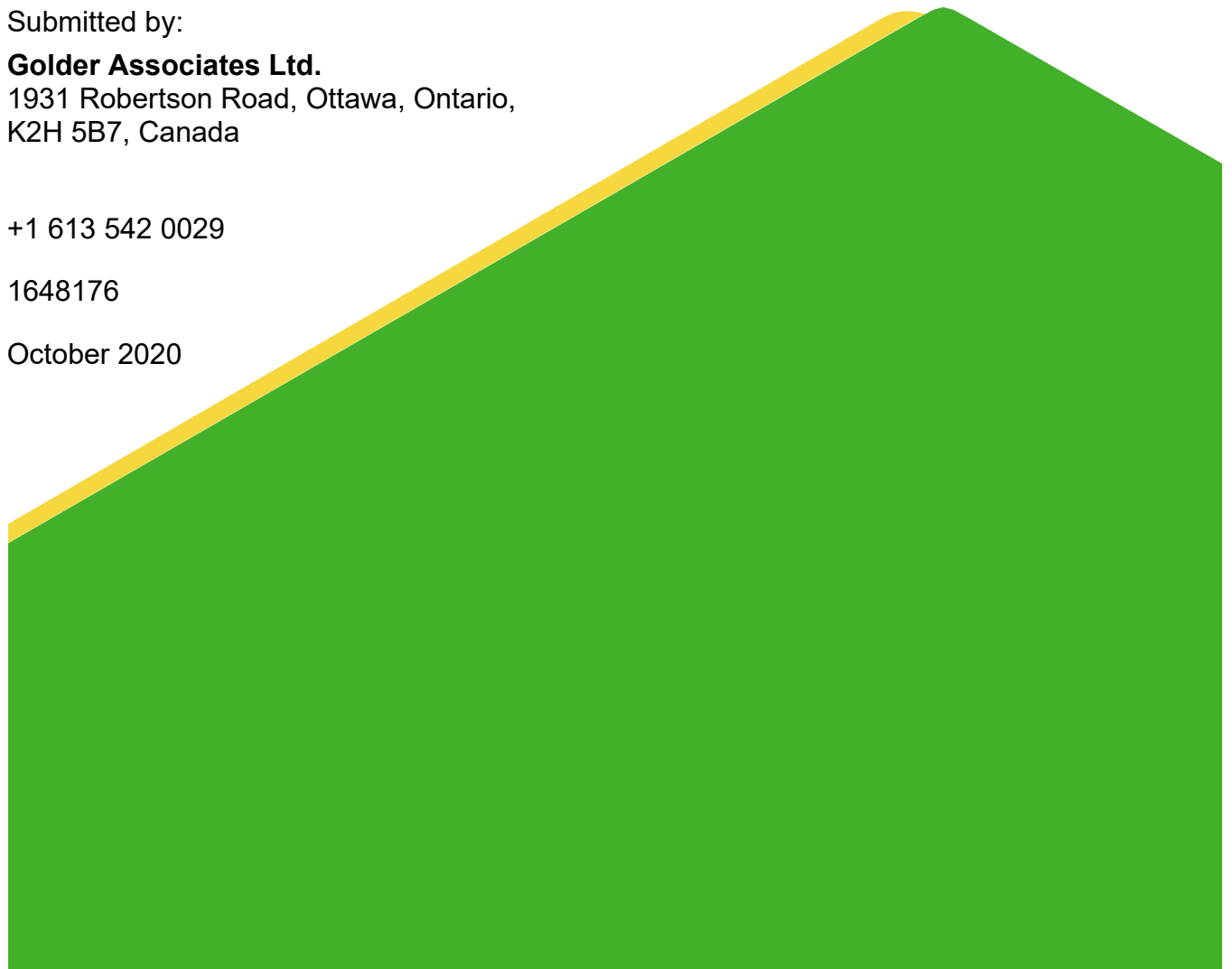
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October 2020



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APPENDICES

APPENDIX A

Amended Environmental Compliance Approval Number 1828-9CKRMK

APPENDIX B

Calculations

APPENDIX C

Model Outputs

APPENDIX D

Pond Hydrographs

Acronyms and Glossary of Terms

Term / Acronym	Definition
AES	Atmospheric Environmental Service
BOD ₅	5-Day Biological Oxygen Demand
CN	Soil Conservation Services Curve Number
CSP	Corrugated Steel Pipe
EA	Environmental Assessment
EASR	Environmental Assessment Study Report
ECA	Environmental Compliance Approval
ESC	Erosion and Sediment Control
ha	Hectare
H:V	Horizontal to Vertical
IA	Initial Abstraction
IDF	Intensity-Duration-Frequency Curve
L/s	Litres per Second
m	Metre
m ³	Cubic metre
masl	Metres Above Sea Level
MECP	Ministry of Environment, Conservation and Parks
MHSW	Municipal Hazardous or Special Waste
MRF	Material Recovery Facility
mm	Millimetre
PWQO	Provincial Water Quality Objective
O.Reg.	Ontario Regulation
SWM	Stormwater Management

Term / Acronym	Definition
ToR	Terms of Reference
t_c	Time of Concentration
t_p	Time to Peak
TSS	Total Suspended Solids
VO2	Visual Otthymo

1.0 INTRODUCTION

The City of London is proposing an expansion of the existing W12A Landfill site (the Project) located at 3502 Manning Drive, part of Lots 18, 19 and 20 of Concession 6 in the former Township of Westminster, in the south end of the City of London, Ontario.

The environment is considered in broad terms to include the natural, social, cultural, and economic aspects of the environment. This document presents the stormwater management component existing conditions and impact assessment for the Environment Assessment (EA) of the proposed W12A Landfill expansion. This work has been conducted in accordance with the requirements set out in the approved Amended Terms of Reference (ToR), dated February 2019.

This Stormwater Management Existing Conditions and Impact Assessment report is a supporting document to the Environmental Assessment Study Report (EASR)

2.0 BACKGROUND

The landfill site opened in 1977 and operates under Environmental Compliance Approval (ECA) A042102 approved by the Ministry of Environment, Conservation and Parks (MECP) under the Environmental Protection Act. The landfill accepts solid non-hazardous waste from its approved service area as well as Municipal Hazardous or Special Waste (MHSW) from residents and small quantity generators. The solid-non-hazardous waste is disposed at the site while the MHSW is sent to licensed facilities for recycling, reuse or disposal.

The purpose of the EA study and proposed expansion of the landfill is to seek approval for additional waste disposal capacity because the existing W12A Landfill is expected to reach its approved total disposal capacity at the end of 2023.

This stormwater management report is based on a vertical expansion of the landfill over the existing footprint as identified as the preferred expansion alternative in the EASR.

2.1 Site Location

The W12A Landfill is located at 3502 Manning Drive in the City of London. The site is located south of Highway 401, bounded to the south by Manning Drive, to the north by Scotland Drive, and on the west by White Oak Road. The landfill is on a surface water divide between Dingman Creek watershed to the north and Dodd Creek watershed to the south.

To accommodate ancillary features, stockpiling of soils and provide additional on-Site buffer compared to the existing landfill, the northern boundary of the landfill property will be extended northward to Scotland Drive and a 100 m buffer width will be provided on the east side of the landfill footprint and some additional area in the southeast corner to tie into the adjacent Material Recovery Facility (MRF) property line.

The overall area of the expanded site is 192.4 ha, but for the purposes of this surface water assessment the focus is on the landfill footprint and the area immediately adjacent where operations will take place (Landfill Operations Area). The majority of the Landfill Operations Area is within the Dodd Creek subwatershed with the remainder within the Dingman Creek

subwatershed, with approximately 142 ha and 13 ha, respectively, of the current landfill site property in each subwatershed (prior to all landfilling activity on the Site).

The soils on site are low permeability clays and groundwater movement is insignificant. Land use in the vicinity of the Site is primarily agricultural.

Five watercourses flow away from the Landfill Operations Area. Four of these are tributaries of Dodd Creek and are located on the south edge of the landfill, starting just north of Manning Drive and crossing the road via culverts. These tributaries combine a few hundred metres south of the Site and continue southwest toward the main branch of Dodd Creek (EarthTech, 2002). The fifth is located west of the landfill discharging via a culvert under White Oak Road and is a tributary of Dingman Creek (EarthTech, 2002).

3.0 EXISTING STORMWATER MANAGEMENT

As noted in Section 2.1, the W12A Landfill is situated on the boundary between two subwatershed areas, Dingman Creek and Dodd Creek. Dingman Creek is a tributary of the Thames River (Lake St. Clair Basin) watershed and Dodd Creek is part of the Kettle Creek (Lake Erie Basin) watershed. The majority of the Landfill Operations Area falls within the Dodd Creek subwatershed and a small northerly portion of the Landfill Operations Area falls within the Dingman Creek subwatershed. Figure 1 provides pre-development drainage areas and surface water features.

Four surface water features crossing Manning Drive convey runoff from the W12A Landfill property to downstream receiving water systems within the Dodd Creek watershed and one additional surface water feature crossing White Oak Road conveys runoff from the site to a tributary of Dingman Creek. All five tributaries are documented as being seasonally dry, intermittent watercourses with a majority of flow generated as a result of direct runoff during precipitation events.

The existing drainage network in the vicinity of the landfill is divided into four general areas with a system of berms, slopes and perimeter drainage ditches directing runoff generated within the W12A Landfill Operations Area to four separate stormwater management (SWM) ponds. Stormwater management ponds 2/3, 4 and 5 are located along the south side of the site and currently service approximately 129.3 ha of drainage area and discharge to the Dodd Creek sub-watershed, see Figure 2. Stormwater management pond 1 is located at the northwest corner of the landfill area and services approximately 14.4 ha of drainage area that falls within the Dingman Creek subwatershed area. The following sections provide an overview of the current stormwater management ponds at the Site. The Site stormwater management facilities operate under ECA 1828-9CKRMK issued by the MECP December 11, 2013, which has been included in Appendix A for reference.

3.1.1 Stormwater Management Pond 1

Stormwater Management Pond 1 is located at the northwest corner of the landfill area and is an extended detention wet pond that receives stormwater runoff from a total drainage area of 14.4 ha under existing conditions. The existing pond was designed to provide Enhanced 80% long-term suspended solids removal as defined by the MECP Stormwater Management

Planning and Design Manual (MECP, 2003) and has a permanent pool storage capacity of 2,470 m³ and an active storage capacity of 5,800 m³. The pond includes a forebay, a permanent pool with elevation 273.2 masl, a 150 mm diameter Hickenbottom outlet structure discharging via a 150 mm diameter gate valve into an existing 900 mm diameter CSP culvert into Tributary 7 and a 3.2 m wide rip-rap overflow at invert elevation 275.0 masl discharging into Tributary 7.

3.1.2 Stormwater Management Pond 2/3

Stormwater Management Pond 2/3 is located in the southwest corner of the Site and is an extended detention wet pond that receives stormwater runoff from a total drainage area of 30.6 ha under existing conditions. The pond was designed to provide Normal 70% long-term suspended solids removal as defined by the MECP Stormwater Management Planning and Design Manual (MECP, 2003) and has a permanent pool storage capacity of 2,660 m³ and an active storage capacity of 11,100 m³. The pond includes a forebay, a permanent pool with elevation 271.2 masl, a 300 mm diameter Hickenbottom outlet structure discharging via a 300 mm diameter gate valve into an existing outlet ditch that drains into Tributary 5, one ditch inlet structure and 750 mm diameter concrete pipe outlet that drains via an outlet ditch into Tributary 5 and one 5.0 m wide overflow spillway at invert elevation 274.0 masl.

3.1.3 Stormwater Management Pond 4

Stormwater Management Pond 4 is located on the south side of the Site and is an extended detention wet pond that receives stormwater runoff from a total drainage area of 57.7 ha. The existing pond was designed to provide Normal 70% long-term suspended solids removal as defined by the MECP Stormwater Management Planning and Design Manual (MECP, 2003) and has a permanent pool storage capacity of 5,200 m³ and an active storage capacity of 20,200 m³. The pond includes a forebay, a permanent pool with elevation 269.7 masl, a 125 mm diameter Hickenbottom outlet structure discharging via a 150 mm diameter gate valve into a 450 mm diameter pipe into an outlet structure with a ditch inlet catchbasin top followed by a 1,050 mm diameter pipe into Tributary 3.

3.1.4 Stormwater Management Pond 5

Stormwater Management Pond 5 is located on the southeast corner of the landfill Site and is an extended detention wet pond that receives stormwater runoff from a total drainage area of 41.0 ha under existing conditions. The pond was designed to provide Normal 70% long-term suspended solids removal as defined by the MECP Stormwater Management Planning and Design Manual (MECP, 2003) and has a permanent pool storage capacity of 3,690 m³ and an active storage capacity of 11,800 m³. The pond includes a forebay, a permanent pool with elevation 268.5 masl, an outlet structure with a 150 mm diameter orifice and an overflow weir at elevation 269.4 masl and a 600 mm x 600 mm slide gate valve.

4.0 PROPOSED EXPANSION CONDITIONS

The existing drainage network in the vicinity of the landfill is currently divided into four general areas with a system of berms, slopes and perimeter drainage ditches directing runoff generated within the W12A Landfill Operations Area to four separate stormwater management ponds. Under the proposed vertical expansion, the peak of the landfill mound will shift to the south and sideslopes of the mound will be longer and somewhat steeper in places, resulting in changes to the drainage boundaries to each of the ponds and, in general, reduced times to peak runoff. In addition, a proposed ancillary facility including public drop off areas, access roads, bins, and storage areas is proposed to be added east of Pond 5. The post-expansion, post-closure drainage areas are shown on Figure 3. To address these changes in surface water runoff, some modifications to the existing stormwater management ponds will be required.

Under current conditions, Ponds 2/3, 4 and 5 collect an area of 129.3 hectares of surface water runoff and provide treatment and attenuation prior to discharge off-Site to the south, to the Dodd Creek subwatershed. The proposed landfill expansion and mound configuration will result in a reduction of the drainage area being collected and treated by these ponds and discharged to the Dodd Creek subwatershed to 106.5 hectares, including an additional drainage area to Pond 5 from the proposed ancillary facilities. Under current conditions, Pond 1 collects a drainage area of 14.4 hectares of surface water runoff and discharges off-Site to the north, to the Dingman Creek subwatershed. The proposed landfill expansion would result in an increased drainage area of 34.5 hectares to Pond 1. The outlet structure for Pond 1 will be designed such that discharge will match or be below pre-expansion peak flows for the 1:5 year through 1:100 year design storm events.

The proposed modifications to the stormwater management system will all occur within the Site, specifically to the stormwater management ponds and their outlet structures, and there is no proposed work on the downstream stormwater management infrastructure including the ditches, culverts or the downstream off-Site watercourses themselves. As such, the proposed expansion will not involve physical alterations to downstream watercourses. Also, the ponds will provide Enhanced Level Protection (80% Total Suspended Solids (TSS) removal) and post-development flows are designed to match pre-development flows using the controls in the outlet structures for the stormwater management ponds.

4.1 Design Criteria

In addition to design of the stormwater management system as required by O.Reg. 232/98, a stormwater management report for waste processing sites is required for sites that include provisions for the outdoor storage of waste or discharge from inside of the building to the outside. Since the outdoor storage of waste occurs at the Site, the Site is therefore subject to the stormwater management reporting requirements of a waste processing site as outlined in the *MECP Guide for Applying for an Environmental Compliance Approval* (December 2012) (the Guide).

A stormwater management report is used to assess the potential off-site impacts. According to the Guide, at a minimum, the plan must include a detailed plan of stormwater management, including:

- A description of the nature of the interaction of the waste with rainwater, which should include considerations of volumes of stormwater runoff and storage.
- Surface elevations indicating the direction of drainage and a description of all discharge locations.
- Impact prevention and monitoring plans, if required.

Modifications to the stormwater management ponds will meet the following design criteria where possible:

- Match post-expansion outlet flows to corresponding pre-expansion flows for the 1:5 year through the 1:100 year return period design storm.
- Provide Enhanced Level Protection (80% TSS removal) as defined by the MECP SWM Planning and Design Manual (MECP, 2003).
- Surface drainage from potentially contaminated areas, i.e., originating from active landfilling areas, will be contained locally within berms and will discharge into the waste and eventually into the leachate management system. Surface drainage from non-contaminated areas such as road areas and areas with interim or final landfill cover will be conveyed to the stormwater management pond via the internal drainage ditches.
- Ditches will be sized to convey the 1:100 year return period design storm and culverts sized to convey a 1:25 year return period design storm as per O.Reg. 232/98.
- The 1:250 year return period design storm will also be assessed to predict conditions during such an event to evaluate potential climate change effects.

4.2 Methodology

Runoff scenarios were assessed with the Visual Otthymo (VO2) model.

To estimate peak site runoff rates for the existing Site, it was modelled using the Nash instantaneous unit hydrograph method. This method uses the Soil Conservation Services Curve Number (CN). The CN values were selected to be consistent with the previous Stormwater Master Plan (EarthTech, 2002) and updated based on the proposed increase to landfill sideslopes and addition of the ancillary services area east of Pond 5. The CN values and Initial Abstraction (IA) values used for the model were weighted based on surface type as shown in the following Table.

Surface Type	CN	IA (mm)
Building/Asphalt/ Concrete/Pond	98	1.5
Gravel/Recycled Asphalt	89	3
Landscaped/Landfill top area	81	3
Landfill sideslopes	83	1.5

A modified Chicago distribution was used for various rainfall events from 25 mm and 1:2 through 1:100 year return period design storms. A 4-hour duration Chicago Storm distribution was used for the 1:2 year through 1:100 year return period storms using IDF curves from the AES rainfall gauge at the London Airport.

The estimated Time of Concentration (t_c) was calculated using the Airport Method for catchments with a runoff coefficient less than 0.40 and the Bransby-Williams Equation for catchments with a runoff coefficient greater than 0.40 (based on the weighted catchment Rational Method runoff coefficient C). The Time to Peak (t_p) used in the model was calculated as $t_p = 0.67t_c$. The vertical expansion proposed for the landfill will result in larger areas of steeper sections with 4 horizontal to 1 vertical sideslopes and a top slope of 20 horizontal to 1 vertical. The result of this change will be a shorter time to peak and increased surface water runoff.

The calculations are provided in Appendix B and model outputs are provided in Appendix C and summarized in the following Table 1. The pre-development and current condition inputs are from the previous Stormwater Master Plan (EarthTech, 2002).

Table 1: Model Inputs

Drainage Area ID / (SWM Pond)		Area (ha)	CN	Time to Peak (hours)	IA (mm)
101 (Pond 1)	Pre-Development	14.50	81.0	0.31	6.0
201 (Pond 1)	Current	14.40	81.0	0.25	1.5
301 (Pond 1)	Proposed	34.48	82.4	0.30	2.6
102 (Pond 2/3)	Pre-Development	16.60	81.0	0.39	6.0
103 (Pond 2/3)	Pre-Development	13.00	81.0	0.34	6.0
202/203 (Pond 2/3)	Current	30.60	81.0	0.27	2.2
302 (Pond 2/3)	Proposed	18.47	83.8	0.13	2.2
303 (Pond 2/3)	Proposed	6.24	83.9	0.11	2.3
104 (Pond 4)	Pre-Development	52.00	81.0	0.53	6.0
204 (Pond 4)	Current	57.70	81.0	0.37	1.5
304 (Pond 4)	Proposed	25.17	83.4	0.16	2.2
105 (Pond 5)	Pre-Development	46.00	81.0	0.56	6.0
205 (Pond 5)	Current	41.00	81.0	0.39	1.5
305 (Pond 5)	Proposed	56.60	82.8	0.52	1.5

4.3 Baseflow

The Earth Tech Stormwater Management Plan (EarthTech, 2002) recommended that baseflow storage be provided to stormwater management Ponds 1 and 2/3. EarthTech reviewed continuous rainfall records over a characteristic 13 year period in London and estimated that to provide 5 to 10 L/s baseflow for 70% of the time or more, a storage volume of 4,000 m³ was required for stormwater management Pond 1 and 6,000 m³ was required for stormwater management Pond 2. The proposed design will also include recommended baseflow storage volumes greater than or equal to these volumes.

Pond 5 was not provided with baseflow as it discharges to an enclosed municipal drain, consistent with the existing design.

Pond 4 was not provided with baseflow volume due to space constraints of this pond and as it was noted in the Earth Tech Stormwater Management Plan that there was little benefit in providing baseflow to the headwaters of the channel as opposed to farther downstream where the separate tributaries come together (EarthTech, 2002). This is consistent with the existing design.

4.4 Stormwater Management Ponds

The overall stormwater management system proposed for the expansion is shown on Figure 4.

4.4.1 Pond 1

Stormwater management Pond 1 is located in the northwest corner of the Operations Area and is an extended detention wet pond that currently receives stormwater runoff from a total drainage area of 14.4 ha and a proposed increased drainage area of 34.48 hectares. The pond discharges to the Dingman Creek subwatershed. The existing Pond has a permanent pool storage capacity of 2,470 m³ and an active storage capacity of 5,800 m³. Proposed modifications to Pond 1 are shown on Figure 5 and discussed below.

Due to the increased drainage area and reduction in time to peak resulting from the landfill expansion, the pond will be expanded to the north, increasing the size of the main pond and forebay. The outlet structure will be modified to match or be less than pre-development flows resulting from the smaller pre-development area of that was initially directed to the Dingman Creek subwatershed.

A baseflow storage volume of 4,520 m³ is provided as stacked storage as noted in Section 4.3 and is therefore in addition to the active storage for the 25 mm and 1:5 year through 1:250 year return period design storms (i.e., the baseflow is in addition to the storage provided for these events). A minimum sized orifice of 75 mm will be used to control the pond outflow for the baseflow storage and 25 mm storm, which provides a 150-hour retention time for runoff produced by a 25-mm rainfall event with a 4-hour modified Chicago distribution as shown in the hydrograph for Pond 1 provided in Appendix D. This calculation assumes that the baseflow storage is full at the time of the 25 mm storm event. The new outlet pipe for the pond is designed as a submerged reverse sloped pipe to promote separation/floating of oils, providing potential for spilled material to be recovered prior to off-Site release occurring. The existing outlet structure for the pond will be modified and will be fitted with a valve to allow emergency closure to assist in spill / leachate containment activities, if needed. The resultant average release rate over the extended drawdown period for just the baseflow volume is about 6 L/s. A 600 mm diameter pipe with a ditch inlet grate will be provided at elevation 274.80 masl and also at 275.50 masl to provide discharge control for larger storm events.

Table 3.2 of the MECP Manual provides storage volume design requirements based on specific site imperviousness levels to achieve required TSS removal objectives. Table 3.2 indicates that the minimum storage volume should be based on 190 m³/ha, for 80% TSS removal at an impervious level of 55%. For the Pond 1 drainage area of 34.48 ha, this results in a minimum total required pond volume of 6,551 m³, with 40 m³/ha required as extended detention. Therefore, 5,172 m³ and 1,379 m³ are required, at minimum, for the permanent pool and extended detention, respectively. A permanent pool volume of 7,402 m³ is provided in the proposed wet pond, which exceeds the required permanent pool volume. An additional 2,253 m³ of extended detention storage is provided for the 25 mm design storm. Detailed calculations are provided in Appendix B and model outputs are provided in Appendix C.

The following table provides the design values for the wet pond and compares these values to the minimum or preferred criteria as per Table 4.6 of the MECP Manual:

Table 2: Pond 1 Extended Detention Wet Pond – MECP Design Criteria

Design Element	Design Value	Comparison to MECP Criteria
Drainage Area	34.48 ha	Exceeds Preferred Criteria (>10 ha)
Treatment Volume	Permanent Pool – 7,402 m ³ Active Storage (for 25mm event) – 2,253 m ³	Meets Minimum Criteria
Active Storage Detention Time	150 hours	Meets Preferred Criteria (>24 hrs)
Forebay	Depth – 3.0 m	Meets Preferred Criteria (>1.5 m)
	Maximum Area – 12.3% of total permanent pool	Meets Criteria (< 33%)
Length-to-Width Ratio	Overall – 5.6:1	Pond Exceeds Minimum Criteria (minimum 3:1)
	Forebay – 3.5:1	Forebay exceeds minimum criteria (2:1)
Permanent Pool Depth	Mean depth of 1.0 m	Meets Preferred Criteria (Mean depth 1 m – 2 m)
Active Storage Depth	The 25 mm storm event is 0.73 m above the permanent pool or 0.23 m above baseflow volume.	Meets Minimum Criteria (<1.0 m)
Side Slopes	5H:1V	Meets Minimum Criteria of 5H:1V for 3 m around permanent pool
Inlet	600 mm diameter culvert	Exceeds Minimum Criteria of 450 mm
Outlet	75 mm dia. Orifice at 274.00 m 600 mm dia. outlet pipe at 274.80, 1.0% slope 600 mm dia. Outlet pipe at 275.50 m, 1.0% slope	Meets Minimum Criteria
Maintenance Access	No maintenance drawdown pipe provided. Access for backhoes or dredging equipment provided.	Meets Minimum Criteria
Buffer	Not provided	Does not meet Minimum Criteria of 7.5 m above maximum water quality/erosion control water level (industrial site with restricted access)

The following calculations summarize the design requirements of the forebay as per Section 4.6.2 of the MECP Manual:

Minimum Forebay Settling Length

$$Dist = \sqrt{\frac{rQ_p}{V_s}}$$

Where: Dist = forebay length (m)
 r = length-to-width ratio
 Q_p = peak flow rate from the pond during design quality event
 (25 mm storm event) (m³/s)
 V_s = settling velocity (m/s)

$$Dist = \sqrt{\frac{3.5(0.012)}{0.0003}}$$

$$Dist = 11.83 \text{ m}$$

Minimum Dispersion Length

$$Dist = \frac{8Q}{dV_f}$$

Where: Dist = length of dispersion (m)
 Q = inlet pipe capacity (10 year storm event) (m³/s)
 d = depth of permanent pool in the forebay (m)
 V_f = desired velocity in the forebay (m/s)

$$Dist = \frac{8(2.42)}{1.5(0.5)}$$

$$Dist = 25.81 \text{ m}$$

The proposed forebay length is 56.0 metres and is therefore greater than the required lengths for settling and dispersion.

Minimum Forebay Bottom Width

$$Width = \frac{Dist}{8}$$

Where: Dist = greater value of minimum forebay length or length of dispersion (m)

Width = minimum forebay bottom width (m)

$$Width = \frac{25.81}{8}$$

$$Width = 3.23 \text{ m}$$

The proposed bottom width is 16.0 metres (average) and is therefore greater than the required width.

A comparison of pre-development to existing and proposed post-development site discharge rates is provided in the following table for the 25 mm and 1:5 year through 1:100 year return period design storm events. As mentioned above, the release rates presented are conservative and assume the baseflow volume is full prior to the storm event. For example, if the baseflow volume was empty, then the 1:5 year peak release rate would be 10 L/s compared to the 167 L/s noted below.

Table 3: Pond 1 Estimated Pre-development and Proposed Peak Flow Rates

Return Period	Pre-Dev Peak Flow (L/s)	Current Conditions Peak Flow (L/s) ¹	Proposed Peak Flow (L/s)	Active Storage Volume ² (m ³)	Depth Above Perm. Pool (m)	Elevation (masl)
25 mm	78		10	2,253	0.73	274.73
1:5 Year	315	10	167	5,808	1.07	275.07
1:10 Year	434		288	6,999	1.18	275.18
1:100 Year	941	430	560	11,740	1.58	275.58
1:250 Year	1,206		988	15,560	1.89	275.89

Notes: 1 From Earth Tech Stormwater Management Master Plan (Earth Tech, 2002)

2 Does not include baseflow volume of 4,517 m³ (i.e., Stacked Storage)

4.4.2 Pond 2/3

Stormwater management Pond 2/3 is located in the southwest corner of the Site and is an extended detention wet pond that currently receives stormwater runoff from a total drainage area of 29.6 hectares and under proposed conditions would have a smaller drainage area of 24.71 hectares. The pond discharges to the Dodd Creek subwatershed. Proposed modifications to Pond 2/3 are shown on Figure 6 and discussed below.

The existing Pond has a permanent pool storage capacity of 2,660 m³ and an active storage capacity of 11,100 m³. The landfill expansion will result in a decreased drainage area to this pond; however, to meet the Enhanced Level Protection (80% TSS removal) as defined by the MECP Stormwater Management Planning and Design Manual (MECP, 2003) the permanent pool depth will be increased and some modifications will be made to the existing outlet structure.

A baseflow storage volume of 6,320 m³ is provided as stacked storage as noted in Section 4.3 and is therefore in addition to the active storage for the 25 mm and 1:5 year through 1:250 year return period design storms (i.e., the baseflow is in addition to the storage provided for these events). A minimum sized orifice of 75 mm will be used to control the pond outflow for the baseflow storage and 25 mm storm, which provides a 70-hour retention time for runoff produced by a 25-mm rainfall event with a 4-hour modified Chicago distribution as shown in the hydrograph for Pond 2/3 provided in Appendix D and assumes the baseflow storage is full at the time of the 25 mm storm event. The outlet pipe for the pond is designed as a submerged reverse sloped pipe to promote separation/floating of oils, providing potential for spilled material to be recovered prior to off-Site release occurring. The existing outlet structure for the pond will be modified and used and is fitted with a valve to allow emergency closure to assist in spill / leachate containment activities, if needed. The resultant average release rate over the extended drawdown period for the baseflow volume is about 11 L/s. An overflow weir with 1.0 m bottom width, 3H:1V sideslopes with rip-rap covering at invert 273.10 masl is proposed to provide discharge control for larger storm events.

Table 3.2 of the MECP Manual provides storage volume design requirements based on specific site imperviousness levels to achieve required TSS removal objectives. Table 3.2 indicates that the minimum storage volume should be based on 190 m³/ha, for 80% TSS removal at an impervious level of 55%. For the pond drainage area of 24.71 ha, this results in a minimum total required pond volume of 4,673 m³, with 40 m³/ha permitted as extended detention. Therefore, 3,707 m³ and 988.4 m³ are required, at minimum, for the permanent pool and extended detention, respectively. A permanent pool volume of 4,602 m³ is provided in the proposed wet pond, which exceeds the required permanent pool volume. An additional 1,451 m³ of extended detention storage is provided for the 25 mm design storm. Detailed calculations are provided in Appendix B and model outputs are provided in Appendix C.

The following table provides the design values for the wet pond and compares these values to the minimum or preferred criteria as per Table 4.6 of the MECP Manual.

Table 4: Pond 2/3 Extended Detention Wet Pond – MECP Design Criteria

Design Element	Design Value	Comparison to MECP Criteria
Drainage Area	23.26 ha	Exceeds Preferred Criteria (>10 ha)
Treatment Volume	Permanent Pool – 4,602 m ³ Active Storage (for 25mm event) – 1,451 m ³	Meets Minimum Criteria
Active Storage Detention Time	70 hours	Meets Preferred Criteria (>24 hrs)
Forebay	Depth – 3.9 m	Meets Preferred Criteria (>1.5 m)
	Maximum Area – 21.0 % of total permanent pool	Meets Criteria (< 33%)
Length-to-Width Ratio	Overall – 11:1	Pond Exceeds Minimum Criteria (3:1)
	Forebay – 10.7:1	Forebay Exceeds Minimum Criteria (2:1)
Permanent Pool Depth	Mean depth of 1.3 m	Meets Preferred Criteria (Mean depth 1 m – 2 m)
Active Storage Depth	1.31 m for 25 mm storm event (includes 1.1 m for baseflow storage) 1.57 m for 5-year storm event (includes 1.1 m for baseflow storage)	Does not meet Minimum Criteria of <1.0 m due to 1.1 m baseflow depth.
Side Slopes	3H:1V	Does not meet minimum criteria of 5H:1V for 3 m around permanent pool. Existing pond has no public access.
Inlet	2 – 900 mm diameter culverts	Exceeds Minimum Criteria of 450 mm
Outlet	75 mm dia. Orifice at 271.70 masl 1.0 m wide overflow weir with 3V:1H sideslopes	Meets Minimum Criteria
Maintenance Access	No maintenance drawdown pipe provided. Access for backhoes or dredging equipment provided.	Meets Minimum Criteria
Buffer	Not provided	Does not meet Minimum Criteria of 7.5 m above maximum water quality/erosion control water level (industrial site with restricted access)

The following calculations summarize the design requirements of the forebay as per Section 4.6.2 of the MECP Manual:

Minimum Forebay Settling Length

$$Dist = \sqrt{\frac{rQ_p}{V_s}}$$

Where: Dist = forebay length (m)
 r = length-to-width ratio
 Q_p = peak flow rate from the pond during design quality event
 (25 mm storm event) (m³/s)
 V_s = settling velocity (m/s)

$$Dist = \sqrt{\frac{10.7(0.041)}{0.0003}}$$

$$Dist = 38.2 \text{ m}$$

Minimum Dispersion Length

$$Dist = \frac{8Q}{dV_f}$$

Where: Dist = length of dispersion (m)
 Q = inlet pipe capacity (10 year storm event) (m³/s)
 d = depth of permanent pool in the forebay (m)
 V_f = desired velocity in the forebay (m/s)

$$Dist = \frac{8(2.851)}{1.3(0.5)}$$

$$Dist = 35.1 \text{ m}$$

The existing forebay length is 79.6 metres and is therefore greater than the required lengths for settling and dispersion.

Minimum Forebay Bottom Width

$$Width = \frac{Dist}{8}$$

Where: Dist = greater value of minimum forebay length or length of dispersion (m)

Width = minimum forebay bottom width (m)

$$Width = \frac{38.2}{8}$$

$$Width = 4.78 \text{ m}$$

The existing forebay bottom width is 7.1 metres (average) and is therefore greater than the required width.

A comparison of pre-development to existing and proposed post-development site discharge rates is provided in the following table for the 25 mm and 1:5 year through 1:100 year return period design storm events.

Table 5: Pond 2/3 Estimated Pre-development and Proposed Peak Flow Rates

Return Period	Pre-Dev Peak Flow (L/s)	Current Conditions Peak Flow (L/s) ¹	Proposed Peak Flow (L/s)	Active Storage Volume ² (m ³)	Depth Above Perm. Pool (m)	Elevation (masl)
25 mm	143		16	1,604	1.33	273.03
1:5 Year	571	150	277	3,551	1.60	273.30
1:10 Year	783		394	4,384	1.71	273.41
1:100 Year	1,513	730	1,113	6,951	2.04	273.74
1:250 Year	2,172		2,059	8,944	2.27	273.97

Notes: 1 From Earth Tech Stormwater Management Master Plan (Earth Tech, 2002)

2 Does not include baseflow volume of 4,517 m³ (i.e., Stacked Storage)

4.4.3 Pond 4

Stormwater management Pond 4 is located centrally on the south side of the Site and is an extended detention wet pond that currently receives stormwater runoff from a total drainage area of 57.7 hectares. The pond discharges to the Dodd Creek subwatershed. The existing pond has a permanent pool storage capacity of 5,200 m³ and an active storage capacity of 20,200 m³. Under proposed conditions Pond 4 would receive stormwater runoff from a smaller total drainage area of 25.17 hectares. Proposed modifications to Pond 4 are shown on Figure 7 and discussed below.

The existing permanent pool and active storage is sufficiently sized to meet the Enhanced Level Protection (80% TSS removal) as defined by the MECP Stormwater Management Planning and Design Manual (MECP, 2003). The existing outlet structure will be modified with

an appropriately sized orifice to control discharge and the existing double inlet catchbasin would be maintained for larger storm events. An overflow weir would be added for storm events larger than the 1:100 year return period design storm.

The extended detention wet pond outlet structure provides an 80-hour retention time for runoff produced by a 25-mm rainfall event with a 4-hour modified Chicago distribution as shown in the hydrograph for Pond 4 provided in Appendix D. The outlet pipe for the pond for quality control will use the existing perforated 1500 mm Corrugated Steel Pipe (CSP) maintenance hole and internal 200 mm orifice with the Hickenbottom riser structure removed. The existing outlet structure for the pond will be modified and used and is fitted with a valve to allow emergency closure to assist in spill / leachate containment activities, if needed. A 600 mm outlet pipe with a ditch inlet grate will be provided at elevation 270.40 masl for the controlled discharge of large storm events.

Table 3.2 of the MECP Manual provides storage volume design requirements based on specific site imperviousness levels to achieve required TSS removal objectives. Table 3.2 indicates that the minimum storage volume should be based on 190 m³/ha, for 80% TSS removal at an impervious level of 55%. For the pond drainage area of 25.17 ha, this results in a minimum total required pond volume of 4,782 m³, with 40 m³/ha permitted as extended detention. Therefore, 3,776 m³ and 1,007 m³ are required, at minimum, for the permanent pool and extended detention, respectively. A permanent pool volume of 4,387 m³ is provided in the proposed wet pond, which exceeds the required permanent pool volume. An additional 1,479 m³ of extended detention storage is provided for the 25 mm design storm. Detailed calculations are provided in Appendix B and model outputs are provided in Appendix C.

The following table provides the design values for the wet pond and compares these values to the minimum or preferred criteria as per Table 4.6 of the MECP Manual.

Table 6: Pond 4 Extended Detention Wet Pond – MECF Design Criteria

Design Element	Design Value	Comparison to MECF Criteria
Drainage Area	25.17 ha	Exceeds Preferred Criteria (>10 ha)
Treatment Volume	Permanent Pool – 4,387 m ³ Active Storage (for 25mm event) – 1,479 m ³	Meets Minimum Criteria
Active Storage Detention Time	80 hours	Meets Preferred Criteria (>24 hrs)
Forebay	No forebay provided.	Does not meet minimum criteria. Pond is existing with limited space available.
Length-to-Width Ratio	Overall – 2.83:1	Does not meet minimum criteria of 3:1. Existing pond with limited space available.
Permanent Pool Depth	Mean depth of 1.0 m	Meets Preferred Criteria (Mean depth 1 m – 2 m)
Active Storage Depth	0.30 m for 25 mm storm event 0.78 m for 5-year storm event	Meets Minimum Criteria (<1.0 m)
Side Slopes	3H:1V	Does not meet minimum criteria of 5H:1V for 3 m around permanent pool. Existing pond with no public access.
Inlet	2 – 900 mm diameter culverts	Exceeds Minimum Criteria of 450 mm
Outlet	200 mm dia. Orifice at 269.40 masl 600 mm dia. outlet pipe at 270.40 masl, 1.0% slope	Meets Minimum Criteria
Maintenance Access	Maintenance drawdown pipe provided. Access for backhoes or dredging equipment provided.	Meets Minimum Criteria
Buffer	Not provided	Does not meet Minimum Criteria of 7.5 m above maximum water quality/erosion control water level (industrial site with restricted access)

A comparison of pre-development to existing and proposed post-development site discharge rates is provided in the following table for the 25 mm and 1:5 year through 1:100 year return period design storm events.

Table 7: Pond 4 Estimated Pre-development and Proposed Peak Flow Rates

Return Period	Pre-Dev Peak Flow (L/s)	Current Conditions Peak Flow (L/s) ¹	Proposed Peak Flow (L/s)	Storage Volume (m ³)	Depth Above Perm. Pool (m)	Elevation (masl)
25 mm	200		34	1,479	0.37	269.67
1:5 Year	770	80	68	4,569	0.84	270.14
1:10 Year	1,050		78	5,959	1.04	270.34
1:100 Year	2,013	1,800	363	9,225	1.47	270.77
1:250 Year	2,868		593	11,954	1.80	271.10

Notes: ¹ From Earth Tech Stormwater Management Master Plan (Earth Tech, 2002)

4.4.4 Pond 5

Stormwater management Pond 5 is located in the southeast corner of the landfill Site and is an extended detention wet pond that currently receives stormwater runoff from a total drainage area of 41.0 hectares. The pond discharges to the Dodd Creek subwatershed. The existing Pond has a permanent pool storage capacity of 3,690 m³ and an active storage capacity of 11,800 m³. Proposed modifications to Pond 5 are shown on Figure 8 and discussed below.

Under proposed conditions Pond 5 would receive stormwater runoff from a larger total drainage area of 56.60 hectares. The Pond would be expanded to the east and north to increase both the permanent pool and active storage capacities. The existing outlet structure orifice and weir will be modified to match pre-development peak flows.

The extended detention wet pond outlet structure provides a 140-hour retention time for runoff produced by a 25-mm rainfall event with a 4-hour modified Chicago distribution as shown in the hydrograph for Pond 5 provided in Appendix D. The existing outlet structure for the pond will be modified and used and is fitted with a valve to allow emergency closure to assist in spill / leachate containment activities, if needed. The outlet structure includes a 150 mm diameter orifice at elevation 268.50 masl and a 1.2 m wide weir at elevation 269.40 masl.

Table 3.2 of the MECP Manual provides storage volume design requirements based on specific site imperviousness levels to achieve required TSS removal objectives. Table 3.2 indicates that the minimum storage volume should be based on 190 m³/ha, for 80% TSS removal at an impervious level of 55%. For the pond drainage area of 56.60 ha, this results in a minimum total required pond volume of 10,754 m³, with 40 m³/ha permitted as extended detention. Therefore, 8,490 m³ and 2,264 m³ are required, at minimum, for the permanent pool and extended detention, respectively. A permanent pool volume of 12,098 m³ is provided in the proposed wet pond, which exceeds the required permanent pool volume. An additional 2,393 m³ of extended detention storage is provided for the 25 mm design storm. Detailed calculations are provided in Appendix B and model outputs are provided in Appendix C.

The following table provides the design values for the wetland pond and compares these values to the minimum or preferred criteria as per Table 4.6 of the MECP Manual:

Table 8: Pond 5 Extended Detention Wet Pond - MECP Design Criteria

Design Element	Design Value	Comparison to MECP Criteria
Drainage Area	56.60 ha	Exceeds Preferred Criteria (>10 ha)
Treatment Volume	Permanent Pool – 12,098 m ³ Active Storage (for 25mm event) – 2,393 m ³	Meets Minimum Criteria
Active Storage Detention Time	140 hours	Meets Preferred Criteria (>24 hrs)
Forebay	Depth – 2.0 m	Meets Preferred Criteria (>1.5 m)
	Maximum Area – 10.7% of total permanent pool	Meets Criteria (< 33%)
Length-to-Width Ratio	Overall – 4.6:1	Pond Meets Preferred Criteria (from 4:1 to 5:1)
	Forebay – 3.2:1	Forebay exceeds minimum criteria (2:1)
Permanent Pool Depth	Mean depth of 1.5 m	Meets Preferred Criteria (Mean depth 1 m – 2 m)
Active Storage Depth	0.26 m for 25 mm storm event 0.90 m for 5-year storm event	Meets Minimum Criteria (<1.0 m)
Side Slopes	3H:1V	Does not meet minimum criteria. No public access to landfill and pond location.
Inlet	Open ditch	Exceeds Minimum Criteria of 450 mm
Outlet	150 mm dia. orifice 1.2 m wide overflow weir 900 mm dia. outlet pipe with 1.0% slope	Meets Minimum Criteria
Maintenance Access	No maintenance drawdown pipe provided. Access for backhoes or dredging equipment provided.	Meets Minimum Criteria
Buffer	Not provided	Does not meet Minimum Criteria of 7.5 m above maximum water quality/erosion control water level (industrial site with restricted access)

The following calculations summarize the design requirements of the forebay as per Section 4.6.2 of the MECP Manual:

Minimum Forebay Settling Length

$$Dist = \sqrt{\frac{rQ_p}{V_s}}$$

Where: Dist = forebay length (m)
 r = length-to-width ratio
 Q_p = peak flow rate from the pond during design quality event
 (25 mm storm event) (m³/s)
 V_s = settling velocity (m/s)

$$Dist = \sqrt{\frac{3.2(0.020)}{0.0003}}$$

$$Dist = 14.60 \text{ m}$$

Minimum Dispersion Length

$$Dist = \frac{8Q}{dV_f}$$

Where: Dist = length of dispersion (m)
 Q = inlet pipe capacity (10 year storm event) (m³/s)
 d = depth of permanent pool in the forebay (m)
 V_f = desired velocity in the forebay (m/s)

$$Dist = \frac{8(2.42)}{1.5(0.5)}$$

$$Dist = 25.81 \text{ m}$$

The proposed forebay length is 48.0 metres and is therefore greater than the required lengths for settling and dispersion.

Minimum Forebay Bottom Width

$$Width = \frac{Dist}{8}$$

Where: Dist = greater value of minimum forebay length or length of dispersion (m)

Width = minimum forebay bottom width (m)

$$Width = \frac{25.81}{8}$$

$$Width = 3.23 \text{ m}$$

The proposed bottom width is 15.0 metres and is therefore greater than the required width.

A comparison of pre-development to existing and proposed post-development site discharge rates is provided in the following table for the 25 mm and 1:5 year through 1:100 year return period design storm events.

Table 9: Pond 5 Estimated Pre-development and Proposed Peak Flow Rates

Return Period	Pre-Dev Peak Flow (L/s)	Current Conditions Peak Flow (L/s) ¹	Proposed Peak Flow (L/s)	Storage Volume (m ³)	Depth Above Perm. Pool (m)	Elevation (masl)
25 mm	170		20	2,393	0.26	268.76
1:5 Year	654	150	44	8,813	0.90	269.40
1:10 Year	892		242	10,866	1.10	269.60
1:100 Year	1,705	1,400	1,204	15,803	1.54	270.04
1:250 Year	2,427		2,220	19,763	1.88	270.38

Notes: ¹ From Earth Tech Stormwater Management Master Plan (Earth Tech, 2002)

5.0 IMPACT ASSESSMENT

Surface water quantity and surface water quality were examined based on the design criteria provided in Section 4.1. The proposed condition model results were compared with pre-development results, with consideration of proposed design mitigation measures, to determine the “net effect” of the proposed W12A Landfill expansion; these “net effects” are discussed in the following sections.

5.1 Modifications to Existing SWM Infrastructure

As noted in Section 4.0, the proposed modifications to the stormwater management system will all occur within the Site, specifically to the stormwater management ponds and their outlet structures, and there is no proposed work on the downstream stormwater management infrastructure including the ditches, culverts, or the downstream off-Site watercourses themselves. As such, the proposed expansion will not involve physical alterations to downstream watercourses. Also, because of the proposed Enhanced Level Protection (80%

TSS removal) and post-development flows matching pre-development flows due to the controls in the outlet structures for the stormwater management ponds, there is not expected to be an adverse impact on off-Site surface water quantity or quality. The design of Ponds 2/3, 4 and 5 to provide an improved 80% TSS removal, compared to 70% TSS removal in the existing pond design, should result in an overall improvement in the quality of surface water released from these ponds and an overall improvement in surface water quality in receiving watercourses south of Manning Drive.

Pond 1 is currently designed for 80% TSS removal and will be expanded to remain at the same removal level for the larger drainage area and will provide attenuation of flows with post-development flows matching pre-development flows for the 1:5 through 1:100 year return period design storm. As such it is expected that surface water quality to the receiving watercourse to the north (Dingman Creek tributary) will remain the same as current conditions.

5.2 Surface Water Quantity

Since the proposed Project has the potential for effects on surface water management, predicted impacts were assessed with consideration of design mitigation measures. Several mitigation measures are incorporated into the conceptual Site design to manage surface water quantity and minimize potential off-Site impacts. Mitigation options were assessed in the VO2 model, which was used to predict changes in water quantity.

5.2.1 Predicted Changes in Drainage Areas

As noted in previous sections, the existing drainage network in the vicinity of the landfill is currently divided into four general areas with a system of berms, slopes and perimeter drainage ditches directing runoff generated within the W12A Landfill Operations Area to four separate stormwater management ponds. Under the proposed vertical expansion, the peak of the landfill mound will shift to the south and sideslopes of the mound will be longer and somewhat steeper in places, resulting in changes of the drainage boundaries to each of the ponds and, in general, reduced times to peak runoff. In addition, a proposed ancillary facility including public drop off areas, access roads, bins, and storage areas is proposed to be added east of Pond 5. To address these changes in surface water runoff, some modifications to the existing stormwater management ponds are proposed. The proposed expansion conditions scenario considers the Site layout for the ultimate build-out of the London landfill, including final cover, with the stormwater management controls shown on Figure 4.

Dingman Creek

The proposed area of drainage to the Dingman Creek subwatershed from the Operations Area, including Pond 1, will increase by 20.08 ha, to a total area of 34.48 ha. The proposed expansion conditions drainage area includes primarily landfill with some access roads, a stormwater management pond, and surrounding berms.

The Dingman Creek subwatershed overall is 17,200 ha consisting of approximately 30 tributaries to the Creek with the main channel of Dingman Creek converging with the Thames River near Delaware (Delcan, 2005). The increase of 20.08 ha to the subwatershed represents a 0.12% increase to the overall subwatershed area and is in the upper reaches of the subwatershed

adjacent to the divide between the Dingman Creek and Dodd Creek subwatersheds. As such, the change in drainage area is expected to have negligible effect on runoff and drainage within areas of the subwatershed downstream of the W12A site. Further, discharge from the Site to Dingman Creek subwatershed will be attenuated by Pond 1 to below pre-development flows up to and including the 1:100 year return period design storm under the proposed design.

Dodd Creek

The proposed area of drainage to the Dodd Creek subwatershed will include discharge from Ponds 2/3, 4 and 5, which will collect surface water runoff from a development area of 106.48 ha. This area includes the additional drainage area to Pond 5 from the proposed ancillary facilities; however, this area was already within the subwatershed. The net change in subwatershed area is the same 20.08 ha as noted above. The ponds will provide treatment and attenuation prior to discharge off-Site to the south to below pre-development flows up to and including the 1:100 year return period design storm under the proposed design.

The Dodd Creek subwatershed overall is approximately 9,800 ha and is the headwaters to Kettle Creek (Aquafor Beech, 1995). The decrease of 20.08 ha to the subwatershed represents a 0.20% decrease to the overall subwatershed area and is in the upper reaches of the subwatershed adjacent to the divide between the Dingman Creek and Dodd Creek subwatersheds. As such, the change in drainage area is expected to have negligible effect on runoff and drainage within areas of the subwatershed downstream of the W12A Landfill site.

5.2.2 Predicted Effects on On-Site Flows

Post-closure conditions are used for the surface water quantity assessment as the entire Operations Area will be contributing to the Site runoff once the landfill has been capped. The ponds have been sized for the 1:100 year storm event and assessed for the 1:250 year Regional Storm event. Once the modifications to the stormwater management ponds are complete:

- the 1:250 year Regional Storm event peak will be about 45% less than the pre-development peak for the area of the landfill draining to Dodd Creek and less than the 1:100 pre-development peak; and,
- the 1:250 year Regional Storm event peak will be about 20% less than the pre-development peak for the area of the landfill draining to Dingman Creek and marginally greater than the 1:100 pre-development peak.

The following Table provides a summary of the pre-development (prior to any landfilling) and proposed condition peak flows for each subwatershed.

Table 10: Pre-Development and Proposed Expansion Conditions for Subwatersheds

Subwatershed	W12A Site Drainage Areas (ha)		Peak Discharge (L/s)							
			1:5 year		1:10 year		1:100 year		1:250 year	
	Pre	Post	Pre	Post	Pre	Post	Pre	Post	Pre	Post
Dingman Creek	14.5	34.48	315	167	434	288	941	560	1206	988
Dodd Creek	127.6	106.48	1,995	397	2,725	761	5,231	2,482	7,467	4,059

5.3 Surface Water Quality

Stormwater quality control will be provided for the Operations Area to remove a minimum of 80% TSS loading (Enhanced Level Treatment (MECP, 2003)). A detailed assessment of each pond is provided in the Sections above.

To improve settling of TSS within the permanent pool, stormwater management Ponds 1, 2/3 and 5 will include a forebay. Surface water will be directed to the forebay where possible, with the exception of a small area of Pond 5 that cannot be directed to the forebay due to grades, and Pond 4 that does not currently have a forebay and does not have adequate space for one to be provided.

5.4 Predicted Effect on Surface Water Quality

During the operational/construction phase of the Project, ditches and swales at the perimeter of unvegetated portions of the Site will be protected from potential runoff containing suspended solids through the use of temporary berms and silt fences. Perimeter ditches along the completed and capped areas will divert runoff through grass lined swales to the stormwater management ponds.

The ponds and swales will serve to remove suspended sediment from the runoff and to prevent significant outflows that could potentially impair the water quality in downstream watercourses in extreme events.

In the post-closure phase of the Operations Area, finalized perimeter ditches along the outer perimeter of the landfill footprint will capture and direct runoff from the landfill surface and will continue to direct the water via grass lined swales or ditches to the stormwater management ponds. As described in previous Sections, the stormwater management ponds are designed for Enhanced Level Protection (MECP, 2003). The proposed works are predicted to result in surface water quality conditions that are comparable to existing conditions and meet MECP Provincial Water Quality Objective (PWQO) (MOEE, 1994) requirements. Post-closure, the pond operations will continue such that surface water quality downstream of the Site remains protected.

6.0 MONITORING, OPERATION AND MAINTENANCE

6.1 Erosion and Sediment Control

The inspection of Erosion and Sediment Control (ESC) measures during construction should occur on a weekly basis, at minimum. ESC inspection during construction should also occur after significant rainfall events (e.g. greater than approximately 10 mm). An inspection report, highlighting any ESC deficiencies, should be prepared for each inspection, and kept on-Site for reference and reporting purposes, if needed.

Visual inspections of stormwater management or water conveyance features should be performed post-construction on a quarterly (seasonal) basis to ensure sediment build-up has not caused any conveyance capacity issues or potential for an increase in TSS loadings transported downstream. During rainfall-runoff events, visual observations will continue to support the post-development runoff assessment and successful performance of the stormwater management ponds in meeting Enhanced Level of treatment (MOE, 2003).

At minimum, the following should be observed during inspections:

- Signs of erosion of the stormwater management structures. This is important, particularly before the re-vegetation cover has been established.
- Sediment build-up in the swales. For any retention controls (i.e., rock check dams, sediment traps), sediment build-up can be expected at the upstream end of these structures and therefore the stormwater conveyance channels should be inspected on a regular basis and cleaned out periodically to avoid sediment deposits being transported off-Site. Clean-out is recommended to occur once sediment accumulation is clearly visible. In practical terms, clean-out of the rock check dams is recommended if the build-up is greater than one-half the height, from the toe to the spillway. Sediment should be removed in a manner that avoids escape of the sediment downstream and that avoids damage to the control structure. Sediment should be removed to the level of the grade existing at the time the control structure was constructed.
- Ponding in the swales or sediment traps.
- Silt fencing. All silt fences used for ESC should meet required minimum height of 0.6 m. They should be repaired or replaced if damaged.

Environmental monitoring related to surface water at the landfill will be carried out concurrently with the overall Site monitoring program and is described in the main body of this report.

6.2 Surface Water Quality

Surface water quality is regularly monitored as part of a stormwater management discharge monitoring program. Sampling from all stormwater management ponds is undertaken four times per year after significant rainfall events. Sampling occurs at the outfall of Pond 1, Pond 2/3, Pond 4 and Pond 5 in spring and fall and two other sampling events. The samples collected in the spring and fall are analysed for the comprehensive list of parameters and the samples from the other two events are analysed for the indicator parameter list provided in

Table 1 of ECA 1828-9CKRMK. This table includes general parameters, nutrients, major ions, trace metals, volatiles, and field test parameters.

The valves on ponds are normally open. Sampling is compared to trigger values listed in Table 2 of ECA 1828-9CKRMK and provided in the following table:

Parameter	Trigger Concentration (mg/L)
Ammonia (un-ionized)	> 0.3
BOD ₅	> 10.0
Chlorides	> 800
Sulphates	> 1000

In the event of an exceedance of a trigger parameter, additional surface water sampling is conducted at the location of the exceedance to confirm the result. If the second sample results in an exceedance, then the stormwater management pond is operated in batch discharge mode with the gate valve normally closed. Three (3) consecutive surface water sampling events for each trigger parameter exceeded are required to be less than the corresponding trigger concentration for each parameter before the pond can be returned to normal operation (valve open).

Based on review of the existing ECA requirements, it is considered that the existing monitoring program provides sufficient protection for the downstream environment and it is proposed that the existing surface water monitoring program outlined in ECA 1828-9CKRMK be continued for the proposed expansion and modifications to the stormwater management system.

Signature Page

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