Cell 20-1 Lambton Landfill

Cell 20-1, Slope Issues – Clean Harbors Lambton Facility Landfill, Corunna, Ontario

Clean Harbors Canada Inc.

December 17, 2021

→ **The Power of Commitment**

GHD

455 Phillip Street, Unit 100A

Waterloo, Ontario N2L 3X2, Canada

T +1 519 884 0510 | **F** +1 519 884 0525 | **E** info-northamerica@ghd.com | **ghd.com**

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			Name	Signature	Name	Signature	Date
S4	00	Jim Yardley	Dan Puddephatt		Michael Cant		12/17/2021

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1. Introduction

Clean Harbors Canada, Inc. (Clean Harbors) operates a hazardous waste landfill facility (Facility or Site) in Corunna, Lambton County, Ontario. The Facility is located at 4090 Telfer Road (Lots 8 and 9, Concession 10) in St. Clair Township, Lambton County. The Site has a total property area of 140 hectares (ha). The layout of the existing Facility and key features are shown on Figure 1.

This report pertains to the below-grade landfill expansion of the cell identified as Cell 20-1 (Cell). Figure 2 provides the Cell 20-1 general layout and dimensions. Clean Harbors hired Murphy Contracting (Murphy) to carry out the construction of the Cell.

Excavation of the Cell commenced in the spring of 2021. The excavation of the Cell consisted of two excavation zones simultaneously. A bulk excavation zone of the central portion of the Cell and a cell perimeter excavation zone was established that excavated the perimeter of the cell in accordance with the cell design. The Cell design included a ramp down into the base of the cell located along the east wall. The base of the Cell was excavated to the design bottom elevation of 181.5 m AMSL in a south to north direction. Late on the morning of August 18th, Murphy staff observed that a crack was present in the native clay on the west wall in the north-west section. Approximately 45 minutes later, the west side wall started to visually shift / rotate and an upward bulge of west portion of the Cell floor was observed (Slope Issues).

In response to the Slope Issues, MECP issued correspondence to Clean Harbors including an amendment to the Waste Disposal Environmental Compliance Approval (ECA) A031806 as follows:

> *46. The Owner shall not place any waste into the eastern half of cell 19-3 or Cell 20-1 until a report is prepared by a Professional Engineer confirming that landfilling can resume in these cells in a manner that is protective of the health and safety of people and the environment.*

GHD has prepared this report to address the environmental requirement set out in Condition 46, and at the direction of Clean Harbors. The geotechnical and stabilization of the Cell is addressed in a separate report titled "Geotechnical Evaluation and Remedial Plan, Cell 20-1, Slope Issues – Clean Harbors Lambton Facility Landfill Corunna, Ontario" dated November 12, 2021 (Geotechnical Report).

2. Groundwater Modelling – Vertical Landfill Expansion Approval

Extensive groundwater modelling of the Site was conducted previously. The latest groundwater modelling was conducted as part of the assessment of the Site during the Environmental Assessment Act that was submitted to MECP on October 17, 2014 and approved by Order in Council on July 22, 2015 and further advanced to address specific site design components in the Revised Design and Operations Report for the Site prepared by Tetra Tech dated October 18, 2015 and approved by amendment to the Environmental Compliance Approval (ECA) A031806 issued October 19, 2015.

The fundamental basis of the landfill design is based on developing and maintaining an inward groundwater gradient into the landfill waste mound through the installation of a perimeter leachate collection trench and the management of the leachate heads within the trench. The groundwater modelling has been built up through the last 30 years through various assessments and approvals. The modelling conducted with regard to the Environmental Assessment program was a 2-dimensional (2-D) model assessment of various different landfill configurations and comparison to extensive 3-dimensional modelling conducted previously at the Site. The groundwater modelling is documented in the Geology and Hydrogeology Net Effects Analysis and Comparative Evaluation Final Report, Clean Harbors Lambton Landfill Expansion EA prepared by RWDI and released October 2014 (Expansion Report). The purposes of the modelling was to compare the existing landfill with the vertical expansion landfill and demonstrate that the vertical expansion had less impact to the local environment from a groundwater perspective.

The Clean Harbors Lambton Landfill vertical expansion that was approved is referred to in the Expansion Report as Alternative Method 1. The key sections of the Expansion Report that deal with Alternative Method 1 are:

- Section 2.2.1 Alternative Method 1 – Vertical Expansion of Existing Landfill Site
  - Provides key landfill design considerations and assumptions
- Section 3.1.2 Alternative Method 1
  - Provides net effects analysis of the water quantity for alternative method 1 in comparison to the existing landfill.
- Appendix A

Appendix A of this report provides a copy of the key sections of the Expansion Report that relates to Alternative Method 1.

The Expansion Report indicates that the critical parameter from a groundwater impact perspective is the advective transport mechanism and that the diffusive mechanism is relatively minor. As such, the groundwater quantity and groundwater flow movement are the primary component to be considered when comparing the existing (prior to vertical expansion) to the proposed vertical expansion conditions.

The Alternative Method 1 assessment divided the landfill into four areas. Cell 20-1 is located in the area referenced as Cell 16 & 17.

3. Cell 20-1 Assessment and Comparison

3.1 General

The Alternative Method 1 assessment provided in the Expansion Report divided the landfill into four areas. Cell 20-1 is located in the area referenced as Cell 16 & 17.

To assess the environmental impact related to the Slope Issue, review of the groundwater modelling that was conducted for Cell 16 & 17 in 2014 was completed. Based on the review, a pre-Slope Issue for Cell 20-1 could be developed based upon the originally Cell 16 & 17 Alternative Method 1 model. This would allow for a comparison of Cell 20-1 prior to and after the slope issue.

The Slope Issue caused a disturbance to the western interior side slope and to a portion of the base of Cell 20-1. The Geotechnical Report provides geotechnical modelling of the Slope Issue and a remediation program to stabilize the slope through the placement of compacted clay buttress. Additional geotechnical testing was conducted of the Cell 20-1 area and the results of the testing is provided in the report titled “Addendum to Geotechnical Evaluation and Remedial Plan, Cell 20-1, Slope Issues – Clean Harbors Lambton Facility Landfill Corunna, Ontario” dated December 17, 2021 (Addendum).

The Addendum report concludes that the slide path for the slope extended to 175 m ASL. The base of the undisturbed cell is 181.5 m ASL and the base of the clay buttress is 190 m ASL. For assessment of the groundwater assessment the depth of the cell is assumed to be 175 m ASL. The clay soil above 175 m ASL is considered to be compromised and although present has a hydraulic conductivity that is less than the undisturbed clay. For modelling purposes and assessment, the clay above 175 m ASL is assumed to have the same hydraulic conductivity as the waste material. This is a conservative approach since the clay that is present or has been placed as part of the remedial measures has hydraulic conductivity that is lower than the waste.

3.2 Leachate Mounding

Leachate mounding within the waste was assessed as part of the landfill amendment submitted in 2018 and approved by MECP in 2019. The landfill amendment is documented in the report titled “Landfill Design Amendments, Supporting Documentation to Variance to ECA A031806, Clean Harbors Canada, Inc.” prepared by GHD and dated

March 9, 2018. The leachate mounding calculations are provided in Appendix A of the report. Appendix B of this report contains a copy of the leachate mounding calculation tables from the amendment report. Table 1 provides the leachate mounding model developed previously for a base slope of 0.005, 0.1, and 0.5 percent. The 0.5 percent base slope is provided for comparison to the amendment table. Adjusting the equation inputs to reduce the slope to 0.005 percent provides a leachate mound of 1.43 m over a 300 m length or approximately 0.5 m per 100 m of distance from the leachate collection system. For Cell 20-1 the adjusted leachate mound is representative of the leachate mound that is created in a waste mound with a perimeter leachate collection system and no base slope or that waste is located below the leachate collection system.

For Cell 20-1, the length and width of the cell is 158 m by 192 m. For leachate mounding, the waste material is located within 200 m of the perimeter leachate collection system and leachate elevation will be less than 1 m above the maximum operating level of the leachate collection system.

3.3 Advective Flow Assessment

Advective flow is the key parameter at the Site that controls groundwater movement within and adjacent to the landfill. The landfill design and operational parameters were developed to minimize the downward advective flow and to induce flow into the leachate collection system. The perimeter leachate collection was designed to be installed at the perimeter of the existing waste limits and to manage the leachate from the vertical expansion and the existing waste. The perimeter leachate collection system is operated by a pumping system with the pump-ON level established to be at least 1 metre below the groundwater level. The current pump-ON for the leachate collection system is set at 197 m ASL.

Table 2 provides a 1-D analysis of the leakage flux from the base of Cell 20-1 based on the original vertical expansion information, the Cell 20-1 design configuration, the post-Slope Issue configuration, and the post-Slope Issue configuration with the leachate level reduced by one metre.

Under the original vertical expansion modelling the advective flow through the base was downward at a leakage flux rate of 1.8 x 10^{-7} m^3/m^2/day. The Cell 20-1 design has an inward leakage flux rate of 2.4 x 10^{-7} m^3/m^2/day. The Post-Slope Issue Condition has an inward flux rate of 3.5 x 10^{-7} m^3/m^2/day and if the leachate pumping system is reduced by 1 m an inward leachate flux rate of 1.8 x 10^{-6} m^3/m^2/day.

Cell 20-1 is the only vertical expansion landfill cell that is located outside of the pre-vertical expansion waste limit. The original design concept was to continue with the perimeter leachate collection concept design. As a result of the Slope Issue, the perimeter leachate collection was sealed at the southern edge of Cell 20-1. Installing a separate leachate collection system in Cell 20-1 provides an opportunity to control the leachate head within Cell 20-1 separate from the perimeter leachate collection system. The proposed Cell 20-1 leachate collection system concept would be to install leachate collection piping and drainage stone along the south and east perimeter with the base of the leachate collection system located on the upper bench of the cell. Pumping stations would be installed at the three perimeter corners of the perimeter of the cell and connected to the main leachate header system.

The advective flow indicates that the leakage flux is into the Cell 20-1 area for the design after the Slope Issue. With a minor redesign of the leachate collection system for Cell 20-1 and a slight lowering of the pumping elevation, the inward leakage flux can be increased to provide additional environmental control measures without significant increase to the overall leachate generation for Cell 20-1.

3.4 Cell 20-1 Assessment

As part of the Expansion Report modelling, Cell 16 & 17 was modelled as part of the assessment of the chloride movement. The model is provided in Appendix A.2 of the Expansion Report. Figure 3 provides a copy of the model configuration and flow paths from Cell 16 & 17 from the Expansion Report.

The same approach was used to assess the changes in Cell 20-1 pre- and post- Slope Issue. The majority of the thickness and length input information was maintained with adjustments to the elevations, length of LCS and cell area to assess Cell 20-1 only.

Table 3 is setup similar to Table A.2-1 of the Expansion Report. Three scenarios were assessed:

a. Cell 20-1 Design which represents the base condition (Scenario 1),
b. Cell 20-1 Post-Slope Issue with revised base due to disturbance Scenario 2), and
c. Cell 20-1 Post Slope Issue with revised base and a revised leachate collection system (Scenario 3).

The modelling presented on Table 3 indicates that the discharge Cell 20-1 Design (Scenario 1) with the same leachate level used in the Expansion Report has a total discharge of 138.44 m^3/year with the majority of the discharge being to the active aquitard. Scenario 2 where the base has been reduced to reflect the Slope Issue impact elevation of 175 m ASL indicates a small increase in the total discharge to 139.12 m^3/year. The discharge increase is 0.68 m^3/year and is mostly related to an increase in the discharge from the base.

Scenario 3 assessed the proposed redesign of the leachate collection system to be located within the landfill cell, that the leachate collection system would have a lower base and a pumping system that would allow the leachate head in the cell to be lowered. Scenario 3 assumes that the leachate collection system pumping system would be lowered from 198 to 196 mASL and that the leachate mound would be reduced from 198.12 to 197 mASL within the landfill cell. The model indicates that all three flow paths would be reversed from a discharge (outflow) to a inward flow. The predicted inflow would be about 800 m^3/year.

The three scenario flows are provided on Figure 3 for comparison purposes.

4. Conclusion

Based on the review and assessment of environmental aspects discussed in this report, it is concluded that Cell 20-1 can be used for waste disposal based on that the cell can be filled and operated in a manner that is protective of the environment.

As discussed in Section 3.3, the leachate collection system for Cell 20-1 should be re-designed to be installed within the cell at the perimeter and that the leachate collection system operating adjusted accordingly. This matter can be addressed in 2022 and does not prevent waste to filled in the north-west portion of Cell 20-1.

Table 1

Maximum Leachate Head Calculation
Final Cover Installed - No Drainage Layer
Clean Harbors Lambton Facility

		No Drainage Layer		
		0.005% Slope	0.1% Slope	0.5% Slope
Parameters	**Unit**			
conductivity (k)	m/s	1.00E-05	1.00E-05	1.00E-05
	m/year	3.16E+02	3.16E+02	3.16E+02
Drainage length	m	225	225	225
Slope (S)	(dimensionless)	0.005%	0.10%	0.50%
Angle (alpha)	rad	5.0E-05	0.001	0.004999958
L	m	300	300	300
yL	in	0	0	0
	m	0	0	0
YL	(dimensionless)	0	0	0
Parameter				
q	m/yr	0.013	0.013	0.013
q	m/s	4.11945E-10	4.11945E-10	4.11945E-10
R	(dimensionless)	16477.81	41.19	1.65
A	(dimensionless)			
B	(dimensionless)	256.7298635	12.79758655	2.364590249
General case solution (Ymax)				
ymax	m	**1.435329393**	**1.285117304**	**0.880217394**
USEPA solution				
ymax	m	**1.432953737**	**1.251459027**	**0.894458854**

Table 2

Groundwater Flow Calculation Prior to and Following Landfill Rotation
Clean Harbors Canada, Inc.
Lambton Facility

Conditions	Hydraulic Conductivity (m/s)	Groundwater Elevation (mASL)		Elevation (mASL)		Leakage Flux (m^3/m^2 day)(1)(2)	Leakage Discharge(3) (m^3/day)
		Interface Aquifer	Leachate	Top of Interface Aquifer	Base of Waste		
Initial Conditions (2014)	2.6.E-10	198	198.18	159	182	-1.8.E-07	-0.0053
Cell 20-1 Design	2.6.E-10	198	197.75	159	182	2.4.E-07	0.0073
Post-Rotation Conditions	2.6.E-10	198	197.75	159	175	3.5.E-07	0.0105
Post-Rotation Condition - Reduction in Leachate Elevation	2.6.E-10	198	196.75	159	175	1.8.E-06	0.0527

Note:

(1) Leakage Flux is calculated as $q_L = \frac{K \times 86{,}400}{d}(h_1 - h_2)$ from Sptiz and Moreno (1995) Equation 5.10
where:
q_L = Leakage Flux (m^3/m^2 day)
K = Hydraulic Conductivity (m/s)
d = Aquitard Thickness (m)
h_1 = Groundwater Elevation in Interface Aquifer (mASL)
h_2 = Leachate Elevation (mASL)

(2) Direction of leakage flux is vertically upward in both conditions

(3) Based on an assumed cell footprint that is 30,000 m^2

Table 3

Cell 20-1 Groundwater Modelling and Assessment
Lambton Landfill, Clean Harbors Canada Inc.

		Cell 20-1 Design			Cell 20-1 Post-Slope Issue			Cell 20-1 Post Slope Issue and Leachate Collection System Adjustment		
		Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)
Ground Surface Elevation at Toe of Landfill	mASL	200.5	200.5	200.5	200.5	200.5	200.5	200.5	200.5	200.5
Landfill Properties										
Height of Landfill on Closure	mASL	203 to 210	203 to 210	203 to 210	203 to 210	203 to 210	203 to 210	203 to 210	203 to 210	203 to 210
Average Depth of Cell Excavation	m		18.5	18.5		25.5	25.5		25.5	25.5
Landfill Area (A) at Surface	m^2			30,350			30,350			30,350
Exterior Perimeter Length of Landfill at Surface	m	350	350		350	350		350	350	
Active Aquitard										
Total Active Aquitard Thickness	m	6			6			6		
Average Elevation of Interface Aquifer Below Landfill	mASL		159	159		159	159		159	159
Clay Aquitard										
Clay Aquitard Thickness	m			41.5			41.5			41.5
Clay Aquitard Thickness Adjacent to Waste Cell	m		12.5			19.5			19.5	
Hyrdaulic Conductivity - Clay Aquitard	m/s		2.60E-10	2.60E-10		2.60E-10	2.60E-10		2.60E-10	2.60E-10
Engineered Features										
Engineered Cover Thickness	m	2			2			2		
Width of Clay Plug Beyond LCS	m	5			5			5		
Depth of Clay Plug	m	4			4			4		
Anchor Trench Depth	m	3			3			3		
Clay Key Perimeter Width	m	4			4			4		
Hydraulic Conductivity of Weathered Clay Cover/Clay Key	m/s	1.00E-07			1.00E-07			1.00E-07		
Depth of LCS	m	5			5			5		
Invert Elevation of LCS	mASL	195.5			195.5			192		
Parameters Applied In Discharge Calculation										
Water Level Elevation (H/h), expressed in terms of Elevation (mASL)										
Leachte Level in Waste Cell Estimated by Water Balance	mASL	198.12	198.12	198.12	198.12	198.12	198.12	197	197	197
LCS Operating Level	mASL	198			198			196		
Interface Aquifer	mASL		198	198	198	198	198	198	198	198
Distance (L) Along Flow Path to Receptor										
Separation Distance Top of Waste to LCS	m	2	2		2	2		2	2	
Average Separation Distance Waste to LCS via Active Flow Zone	m	3.5			3.5			3.5		
Average Separation Distance Waste to LCS via Less Active Portion of Active Aquitard	m	4.5			4.5			4.5		
Separation Distance between LCS and Ditch	m	8	8		8	8		8	8	
Clay Cap / Key Width	m	3.5			3.5			3.5		
Separation Distance Top of Waste to Ditch	m	10	10		10	10		10	10	
Average Separation Distance Waste to Ditch via Less Active Portion of Active Aquitard	m									
Average Separation Distance Waste to Ditch via Active Aquitard	m									
Average Separation Distance Waste to Ditch via Clay Aquitard	m									
Separation Distance Waste to Interface Aquifer	m		29.25	23.0		25.75	16.0		25.75	16.0
Cross Sectional Area (A) for Groundwater Movement										
Balance of Active Aquitard (lateral flow)	m^2	1267			1267			875		
Clay Aquitard (lateral flow)	m^2		4375			6825			6825	
Landfill Area (vertical flow)	m^2			30,350			30,350			30,350
Hydraulic Conductivity Applied along Flow Path	m/s	0.0000001	2.60E-10	2.60E-10	0.0000001	2.60E-10	2.60E-10	0.0000001	2.60E-10	2.60E-10
Conversion Constant	s/year	31536000	31536000	31536000	31536000	31536000	31536000	31536000	31536000	31536000
$Q = K (\Delta H/L) A$	m^3/year	136.99	0.15	1.30	136.99	0.26	1.87	(788.40)	(2.17)	(15.55)
Total Q Outflow from Landfill Cell	m^3/year	138.44			139.12			(806.13)		

SOURCE:

EXISTING TOPOGRAPHY AND SITE FEATURES PER DRAWING NO. 5, STORMWATER MANAGEMENT, DATED FEBRUARY 23, 2015, PREPARED BY TETRA TECH.

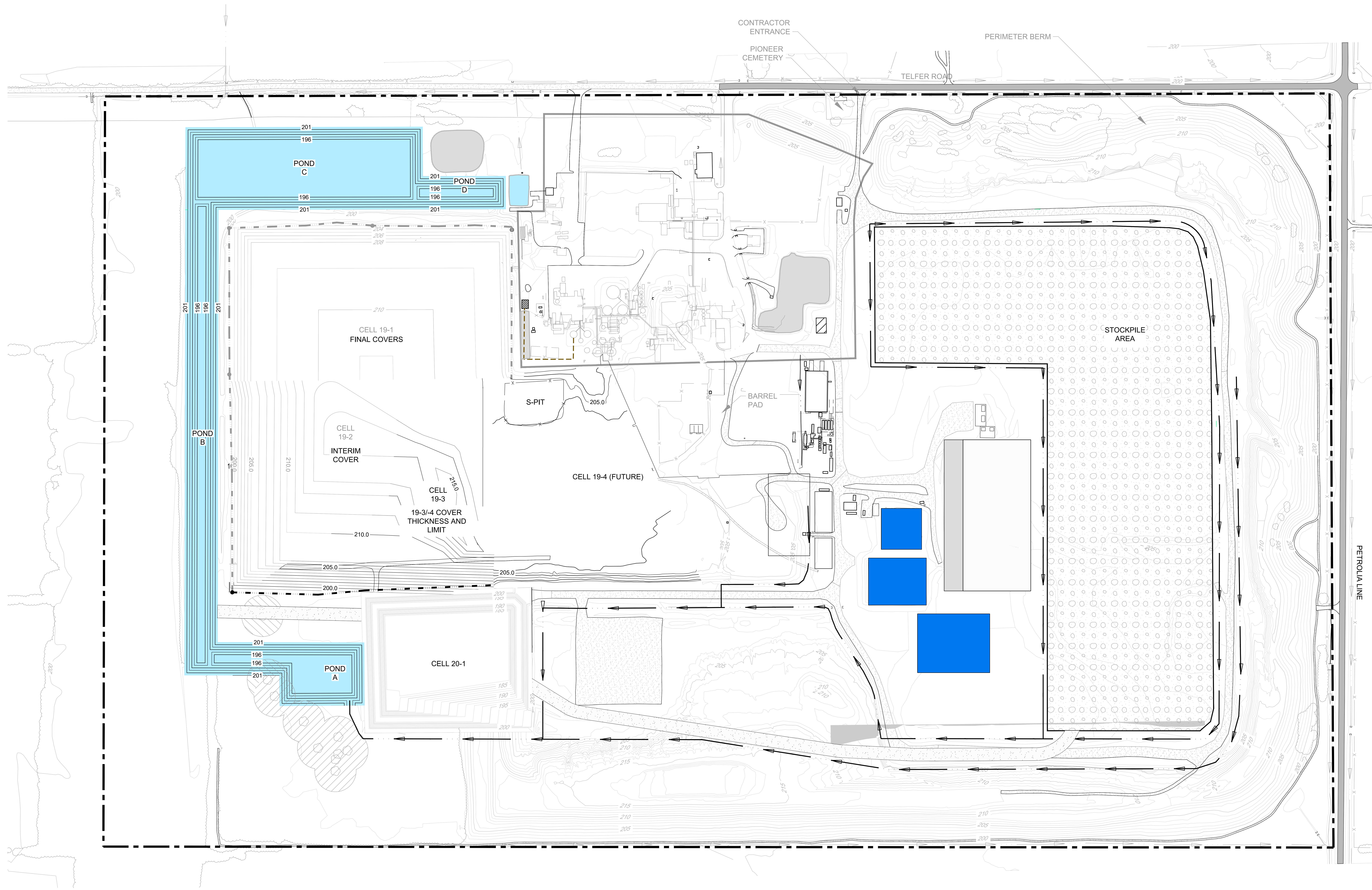
FINAL COVER CONTOURS PER DRAWING NO. 20, FINAL COVER CONTOURS, DATED FEBRUARY 23, 2015, PREPARED BY TETRA TECH.

BUTTERNUT TREES PER DRAWING NO. 13, LANDFILL EXPANSION SUBCELL FILL PROGRESSION PLAN, DATED FEBRUARY 23, 2015, PREPARED BY TETRA TECH.

LEGEND:

- EXISTING MAJOR CONTOUR (5m INTERVAL)
- EXISTING MINOR CONTOUR (1m INTERVAL)
- PROPOSED CONTOUR (2m INTERVAL)
- PROPOSED CONTOUR (1m INTERVAL)
- PROPERTY BOUNDARY
- LIMIT OF WASTE
- LEACHATE COLLECTION SYSTEM
- EXISTING DRAINAGE SWALE
- PROPOSED DRAINAGE SWALE
- FENCELINE
- STORM WATER DRAINAGE DIVIDE
- PAVED ROADWAY
- GRAVEL ROADWAY
- BUILDING / STRUCTURE
- TREELINE
- STORM WATER POND
- PROCESSED WATER
- LEACHATE STAGING POND

0 20 40 60 80m



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www.ghd.com

GHD Ltd.
455 Phillip Street
Waterloo, Ontario N2L 3X2 Canada
T 1 519 884 0510 **F** 1 519 884 0525

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Author	K. DHALIWAL	Designer	-
Drafting Check	-	Design Check	J. YARDLEY
Project Manager	J. YARDLEY	Project Director	

Client

LAMBTON COUNTY, ONTARIO
CLEAN HARBOURS CANADA INC.

Project

2021 CAPITAL WORKS

Date: DECEMBER 16, 2021
Scale: 1:750
Project No.: 44985

Title

GENERAL SITE PLAN KEY FEATURES

Size: ANSI D
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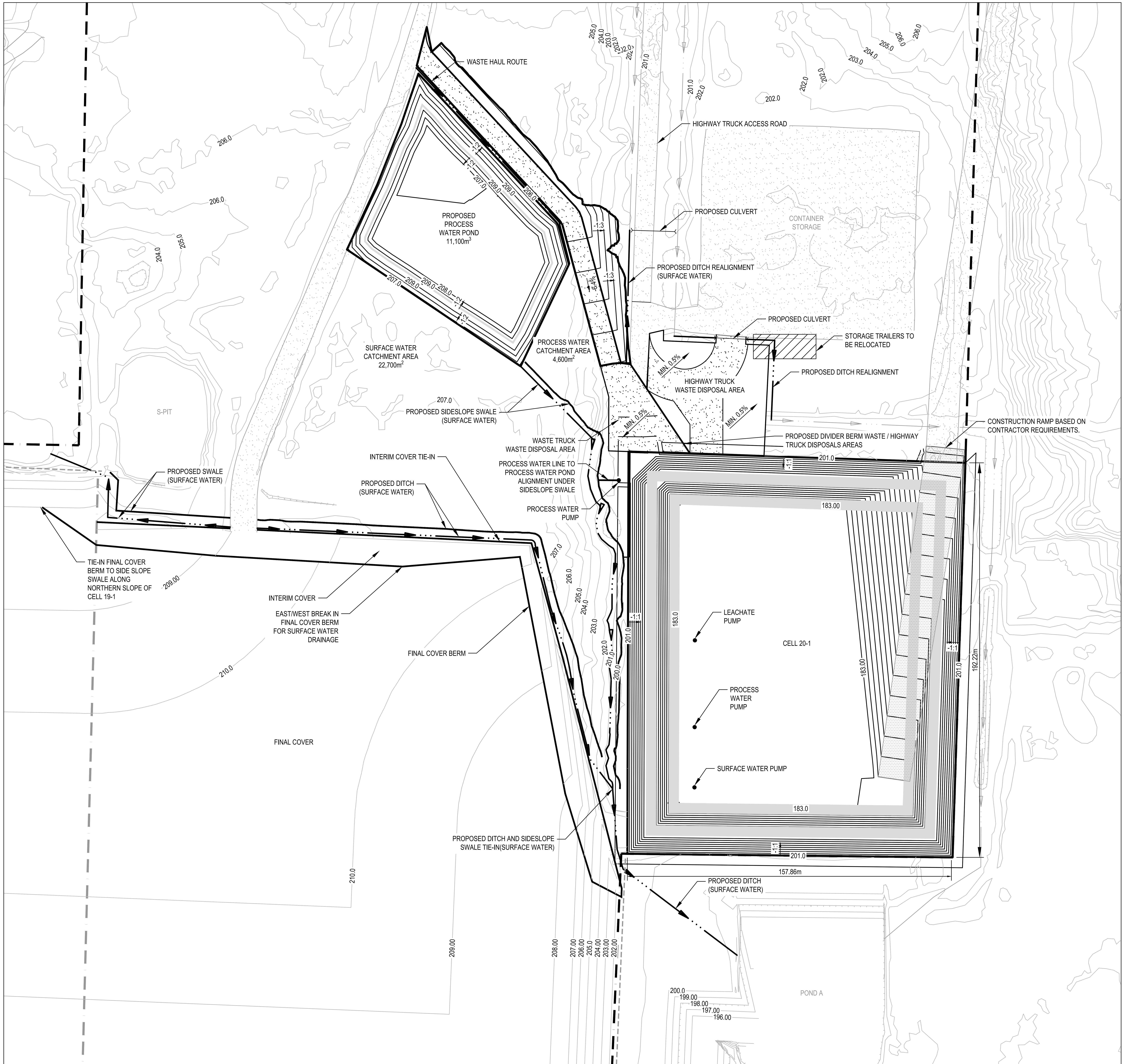
Sheet No.: FIGURE 1
Sheet of

Plot Date: 17 December 2021 - 10:50 AM Plotted By: Spencer Holland Path and Filename: I:\CAD\drawings\44000s\44985\44985-REPORTS\44985-98(051)\44985-98(051)CI-WA001.dwg



LEGEND:

- EXISTING MAJOR CONTOUR (5m INTERVAL)
- EXISTING MINOR CONTOUR (1m INTERVAL)
- PROPOSED MAJOR CONTOUR (5m INTERVAL)
- PROPOSED MINOR CONTOUR (1m INTERVAL)
- LIMIT OF WASTE
- LEACHATE COLLECTION SYSTEM
- EXISTING DRAINAGE SWALE
- EXISTING ROAD / STORAGE AREA
- CONSTRUCTION RAMP
- PROPOSED DITCH/SWALE
- PROPOSED CULVERT
- PROPOSED CLAY BERM
- PROPOSED ACCESS ROAD / DROP OFF AREA

QUANTITES:

CLAY FOR PROCESS WATER POND = 10,100m^3

CLAY FOR BERMS AND DITCHES = 7,300m^3

CLAY FOR ROADS = 2,000m^3

GRANULAR MATERIAL FOR ACCESS ROAD = 1,800m^2

GRANULAR MATERIAL FOR WASTE DISPOSAL AREA= 2,500m^2

FINAL COVER AREA = 76,800m^2

40 mil LINER = 6,500m^2

NOTES:

1. VOLUME FOR "CLAY FOR BERMS AND DITCHES" DOES NOT INCLUDE FINAL COVER BERM ALONG NORTHERN PORTION OF CELLS 19-2 AND 19-3

SOURCE:

EXISTING TOPOGRAPHY FROM AERIAL SURVEY COMPLETED MAY 2020 BY GHD, SUPPLEMENTED WITH SURFACE WATER POND SURVEY FROM RON MURPHY CONTRACTING .

SITE FEATURES COMPILED FROM FOLLOWING SOURCES:
1. AERIAL IMAGERY COMPLETED MAY 2020 BY GHD.
2. LEACHATE TRENCH AND PUMP STATION FROM SURVEY FILES PROVIDED BY RON MURPHY CONTRACTING.

GHD
www.ghd.com

GHD Ltd.
455 Phillip Street
Waterloo, Ontario N2L 3X2 Canada
T 1 519 884 0510 F 1 519 884 0525

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Author	K. DHALIWAL	Designer	-
Drafting Check	-	Design Check	J. YARDLEY
Project Manager	J. YARDLEY	Project Director	

Client

LAMBTON COUNTY, ONTARIO
CLEAN HARBOURS CANADA INC.

Project

2021 CAPITAL WORKS

Date **DECEMBER 16, 2021** Scale **1:1000**

Project No. **44985**

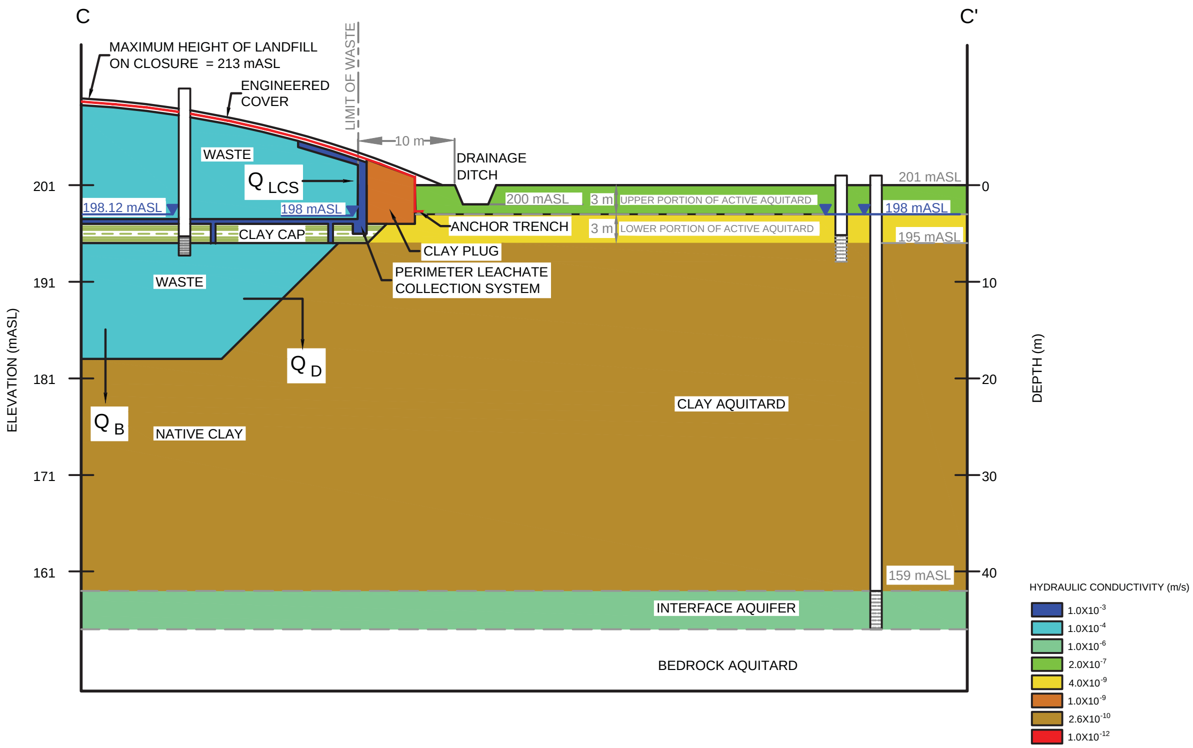
Title

CELL 20-1 AND CELL 19-3 CONSTRUCTION WORKS 2021

Size ANSI D

Status Code

Sheet No. **FIGURE 2** Sheet of



Q_{LCS} = 1.0E-7 x [(198.12 mASL - 198.0 mASL) / 3.5 m] x (5.0 m x 1,106.0 m)
= 597.9 m^3/year

Scenario 1 - 136.99 m3/year (outflow)
Scenario 2 - 136.00 m3/year (outflow)
Scenario 3 - 788.4 m3/year (inflow)

Q_D = 2.60E-10 x [(198.12 mASL - 198.0 mASL) / 29.9 m] x (12.3 m x 1,106.0 m)
= 0.4 m^3/year

Scenario 1 - 0.15 m3/year (outflow)
Scenario 2 - 0.26 m3/year (outflow)
Scenario 3 - 2.17 m3/year (inflow)

Q_B = 2.60E-10 x [(198.12 mASL - 198.0 mASL) / 23.7 m] x (192,139.0 m)
= 8.0 m^3/year

Scenario 1 - 1.30 m3/year (outflow)
Scenario 2 - 1.87 m3/year (outflow)
Scenario 3 - 15.55 m3/year (inflow)

$Q_{(DISCHARGE)}$ = K x (ΔH/L) x *A*

Where:

K = hydraulic conductivity of medium through which groundwater moves;
ΔH = head differential along groundwater flow path [leachate level (H) - water level at receptor (h)];
L = length of groundwater flow path;
A = cross sectional area through which movement occurs.

Figure 3
Groundwater Modelling - Cell 20-1

LEGEND

198.12 mASL ▼ PROJECTED WATER LEVEL
Q_{LCS} SHALLOW LATERAL MIGRATION TO THE PERIMETER LCS
Q_D DEEPER MIGRATION TO INTERFACE AQUIFER
Q_B VERTICAL MIGRATION TO THE INTERFACE AQUIFER

WATER BALANCE
ALTERNATIVE METHOD 1,
CELLS 16 AND 17 BUILD OVER

CLEAN HARBORS CANADA, INC.
LAMBTON FACILITY LANDFILL

Drawn by: TFB	Fig: **A.2-3**
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Appendices

Appendix A

Key Pages from Expansion Report

Clean Harbors Canada, Inc.
Lambton Landfill Expansion Environmental Assessment

Geology & Hydrogeology Net Effects Analysis & Comparative Evaluation Final Report

Prepared by:

CONSULTING ENGINEERS & SCIENTISTS

Tel: 289.205.0618
Fax: 519.823.1316

RWDI AIR Inc.
1100 South Service Road
Suite 324
Hamilton, Ontario, Canada
L8E 0C5
Email: solutions@rwdi.com

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To this end, the effects analysis is to be based on:
  - design considerations and assumptions included in the CDR (including mitigation measures built into the design) that are applicable to the geology/hydrogeology discipline;
  - the results of other technical discipline analysis; and
  - future baseline considerations and assumptions applicable to the geology and hydrogeology discipline.

- the identification of any additional mitigation measures beyond those included in the CDR that can be applied to further minimize or mitigate identified potential environmental effects associated with the landfill alternatives; and

- the analysis of net environmental effects that accommodates the identified mitigation measures.

2.1 Evaluation Criteria & Indicators

The approved ToR set out the assessment criteria and indicators for evaluating the Alternative Methods in the EA. The net effects analysis and comparative evaluation considered two geology/hydrogeology criteria "Groundwater Quality" and "Groundwater Quantity" (**Table 1).**

The Groundwater Quality criterion was selected on the basis that chemical constituents contained in the waste placed at a waste disposal facility will mobilize and move by the processes of advection and diffusion to groundwater receptors, thereby altering groundwater quality in the vicinity of the facility. The effect on groundwater quality at the receptor can be quantified in terms of the potential chemical mass (kilograms/year) discharge or release.

In order to assess this effect, it is initially necessary to develop an understanding of the influence of landfill development on the pattern of groundwater movement locally (namely the advective component of the contaminant transport process). This involves the second criterion, Groundwater Quantity, which recognizes that the excavation of the native clay material to construct landfill cells and the subsequent placement of waste and a cover over the waste will alter the surface runoff and infiltration, and in turn groundwater movement.

During active excavation and for a short period of time while the cells are open and the waste is unsaturated, the precipitation input will be high and groundwater will move from the surrounding overburden into the landfill excavation. Following placement of a clay cap over the waste, most of the precipitation falling on the closed cells will move as runoff to adjacent ditches, and the volume of infiltration entering the cell will be reduced.

In the long-term, the mounding of the water/leachate in the waste will depend on the bounding water levels (i.e., level in the overburden adjacent to and below the landfill), the distance between the waste and the bounding levels, the hydraulic conductivity of the waste and the infiltration rate through the cap (which in turn depends on the hydraulic conductivity of the cap, the slope of the cap and the cover materials on the cap). The hydraulic conductivity of re-compacted clay used in the cap placed over the waste in the landfill cells is expected to be greater (i.e., more permeable) than the hydraulic conductivity of the intact clay overburden in which the cells are excavated.

The rate of infiltration through the cap from precipitation falling on the covered waste will initially exceed outflow through the base and sides of the landfill excavation, resulting in an increase in the liquid level in the waste cell.
Eventually, the level will rise into the cap, reducing the capacity of the cell to accept more infiltration and the water will be rejected as runoff. When the liquid (leachate) level in the waste exceeds the groundwater level in the surrounding overburden, leachate movement will be outward from the waste cells.

Contaminant movement outward from the existing landfill areas is shown conceptually in **Figure 3**. Within the Clean Harbors Lambton Facility property, groundwater is mounded under topographic highs formed by the screening berms located to the perimeter of the landfill and in portions of the existing landfill (e.g., Pre-1986 Landfill) that extend above grade. The shallow groundwater flow path is short, with movement occurring from the topographically high areas to adjacent lows such as ditches and swales. The drainage ditches located internal to the landfill site, collect surface runoff generated on the property and intercept shallow groundwater flow from both the berms and landfill areas.

Depending on surface elevation and the depth of the groundwater table, and the potentiometric pressure in the deep water-bearing zone, referred to as the Interface Aquifer, groundwater movement across the clay aquitard may either be downward (in the situation where the water table elevation is above the elevation of the potentiometric surface) or upward (where the inverse occurs).
Horizontal groundwater movement in the Interface Aquifer is outward from a north south trending potentiometric high below the western part of the property.

The installation of a leachate collection system (LCS) to the perimeter of the landfill, as is intended with the Pre-1986 Landfill area, will further reduce the length of the shallow groundwater flow path adjacent to the landfill. Extraction of liquids from the LCS will also reduce the leachate mounding within the waste and can alter the hydraulic gradient that influences movement outward from the sidewalls and base of the landfill excavation. The effectiveness of the LCS at limiting mounding within the landfill will depend on the degree of hydraulic connection with the waste and the operating water level at the LCS.

Groundwater flow is influenced by the hydraulic gradient and the hydraulic conductivity and porosity of the medium (overburden or bedrock) through which groundwater movement occurs.

The hydraulic gradient (i) is calculated as $i = \frac{\partial \mathrm{H}}{\partial \mathrm{L}}$ where:

Shallow Groundwater and Leachate Movement through the Active Aquitard

∂H = hydraulic head difference between the leachate level in the waste and either the operating level of the LCS, or where a LCS has not been installed, the invert of the nearest perimeter ditch.

∂L = separation distance between the waste to the either the LCS or the nearest perimeter ditch.

Deep Groundwater and Leachate Movement Downward across the Clay Aquitard to the Interface Aquifer

∂H = hydraulic head difference between the leachate level in the waste and potentiometric in the Interface Aquifer.
∂L = separation distance represented by the thickness of clay overburden between the base of the waste cell and the Interface Aquifer.

Depending on the hydraulic head difference, the hydraulic gradient can be either outward/downward from the landfill (indicated by a positive value) or inward/upward into the landfill (indicated by a negative value).

The evaluation of the Groundwater Quantity criterion involves an initial assessment of the potential effect of the landfill design on the vertical and horizontal hydraulic gradients that develop under the proposed designs. The ‘effect’ associated with this criterion is defined as precipitation that comes in contact with the waste to produce leachate, which subsequently moves as a contaminant plume outward from the landfill.

Four pathways for transport of solutes in the leachate are considered:

- Shallow lateral migration through the clay cap/clay key adjacent to the waste and or residual overburden with discharge to the perimeter LCS if present (pathway referred to as QLCS);
- Shallow lateral migration through the shallow overburden (Active Aquitard) with discharge to perimeter ditches (QS);
- Deeper lateral migration from the sidewall of the landfill through a lower portion of the overburden with discharge to the Interface Aquifer (QD); and
- Vertical migration from the base of the landfill with discharge to the Interface Aquifer (QB).

Groundwater and Leachate Movement along Shallow Pathways: The volume of seepage discharge [$Q_{(discharge)}$] expressed in m^3/year, moving outward from the landfill along the shallow flow path to discharge either at a perimeter LCS or on-site surface drainage ditch is calculated as:

$$Q_{(discharge)} = q \text{ x } A$$ where:

q = Darcy flux; and
A = cross sectional area through which solute movement occurs.

Darcy flux (q), which is expressed in metres/second, is calculated as:

$$q = K \text{ x } i$$ where:

K = hydraulic conductivity of medium through which groundwater moves [either the clay cap or weathered/fractured clay overburden (Active Aquitard); and
i = hydraulic gradient (previously defined).

Groundwater and Leachate Movement along Deep Pathways: The volume of seepage discharge [$Q_{(discharge)}$] expressed in m^3/year, moving outward from the landfill along the deep flow path to discharge at the Interface Aquifer is calculated as:

$$Q_{(discharge)} = q \text{ x } A$$ where:

q = Darcy flux; and
A = cross sectional area through which solute movement occurs.

Darcy flux (q), which is expressed in metres/second, is calculated as:

$$q = K \text{ x } i \qquad \text{where:}$$

K = hydraulic conductivity of the un-weathered/un-fractured clay overburden adjacent to and below the base of the waste cell; and
i = hydraulic gradient (previously defined).

The core assumption in the analysis of seepage volumes is that groundwater flow conditions are steady, which is a condition that is expected to develop once the water table and potentiometric pressure at depth below the Facility property stabilize. This condition may take several years to evolve following closure of the landfill.

The analysis approach for the Groundwater Quality criterion is fundamentally similar in that one-dimensional conceptual models are employed. The product of the analysis is an estimate of the potential mass loading of a conservative chemical constituent in the waste (expressed in kilograms/year), to shallow groundwater/surface water internal to the Facility property and to the deep groundwater with movement to the Interface Aquifer that underlies the site.

Chloride was considered in the analysis as a surrogate of the chemical release because: chloride is present as a major constituent of the waste; chloride is highly soluble; and chloride is chemically stable (not degradable or readily attenuated) and therefore mobile. The treatment processes introduced at the Lambton Facility in response to the Land Disposal Restrictions (LDRs) are not expected to significantly alter the chloride mass in the waste.

The advective transport of a chloride from the landfill in the seepage discharge outflow under steady-state flow conditions is calculated as:

Rate of mass discharge $_{\text{(advection)}}$ $= C \text{ x } Q\ (discharge)$ where:

C = chloride concentration in the waste; and
$Q_{\text{(discharge)}}$ = volume of seepage outflow from the landfill calculated for Groundwater Quantity criterion.

Under the process of diffusion, the steady-state mass discharge is proportional to the chloride concentration gradient between the waste and the surrounding environment and is calculated by:

Rate of mass discharge $_{\text{(diffusion)}}$ $= n \text{ x } D^* \text{ x } \frac{\partial C}{\partial Z} \text{ x } A$ here:

n = effective porosity;
D* = effective diffusion coefficient;
$\frac{\partial C}{\partial z}$ = concentration gradient (∂C = change in chloride concentration and ∂Z = distance); and
A = area of landfill across which diffusion occurs.

Over the long-term advection is the dominant process and diffusion becomes an insignificant component of mass discharge. [Note: Diffusive transport was initially considered in the draft Net Effects Analysis & Comparative Evaluation Report (January 2014).

This was based on the assumption that the depth of weathering of the cap would be limited to a 3 m depth and the lower portion of the cap would continue to retain a low hydraulic conductivity. The Township PRT commented that this assumption was not sufficiently conservative (St. Clair Township Peer Review Team, 2014a). As the screening level analysis is intended to be reflective of 'worst case conditions', it was decided to that the analysis be revised. The major change in the core assumptions is that the clay cap would over the long time period considered in the assessment, weather and fracture in a manner similar to the native clay overburden at the Facility. The higher conductivity combined with hydraulic gradients that are outward from the landfill would result in solute transport that is dominated by advection.]

Another significant change that was implemented in the analysis was to reassign solute movement along the deep overburden pathway (QD). It had been assumed in the draft Net Effects Analysis & Comparative Evaluation Report (January 2014) that this discharge would contribute to the chloride loading of the surface water ditches on the property. The MOE in its review of this early draft Report observed that it is possible that not all of the solutes associated with the deeper lateral pathway will discharge to the perimeter ditches. Depending on the water levels in the discharge features, a component of the deeper lateral seepage may actually discharge to the Interface Aquifer.

To assess the relative contribution of discharge to the pathways, groundwater flow under steady-state conditions was evaluated with a numerical simulator MODFLOW-2000 (Harbaugh and others, 2000). Transient solute transport was simulated with MT3DMS (Zheng, 2010). The assessment considered three pathways, namely shallow overburden pathway (QS), deep overburden pathway (QD) and deep overburden pathway with discharge to the Interface Aquifer (QB). The results are presented in a memorandum prepared by SSPA (Appendix D.1). The results of this idealized numerical experiment suggest that with all else being equal, the approach of the screening-level analysis yields an overestimation of the mass discharge to the perimeter ditch and an underestimation of the mass discharge to the Interface Aquifer. The differences between the simplified approach and the more rigorous numerical simulation are however relatively small compared to the conservatism inherent in the screening-level analysis.

Although the results of this assessment indicate that discharge from the deep flow pathway (QB) can be proportioned between surface water and the Interface Aquifer, for the purposes of the screening level calculations herein, the full QD volume is assigned to the Interface Aquifer.

The factors that were considered in estimating the potential long-term chloride mass discharge per the above calculations are:

- the dimensions of the individual landfill cells (area, perimeter length, waste thickness and depth of excavation);
- type (engineered cover system or clay cap), dimensions and hydraulic properties of the landfill cover placed over waste;
- thickness, hydraulic properties (effective porosity and hydraulic conductivity) and coefficient of diffusion for clay overburden adjacent to the landfill and between the base of the landfill and the Interface Aquifer;
- the chloride concentration in the waste leachate; and

- leachate elevation in the landfill, operating level of the leachate collection system, invert elevation of drainage ditches at the toe of the landfill, and shallow and deep groundwater elevations.

The combination of factors will differ for the various landfill designs that have been employed at the existing landfill and for each of the two Alternative Methods that are being considered.

The various factor (parameters) considered in the analysis are discussed in Section 2.2 (Landfill Design Considerations and Assumptions) and Section 2.3 (Future Baseline Conditions and Assumptions).

2.2 Key Landfill Design Considerations & Assumptions

Two conceptual design alternatives were identified in the EA Terms of Reference and are evaluated. These include: Alternative Method 1, a vertical expansion within the footprint of the existing landfill site; and Alternative Method 2, the construction of a new landfill to the south of the existing Facility property. The locations and surface dimensions of the two alternatives are presented in in **Figure 1** and **Figure 2**.

The waste disposal capacity is governed by the limitations imposed on Alternative Method 1 by the size of the disposal area, base grades and top of waste grades. The air space available for disposal is constrained to 3.95 million m^3. Allowing for the placement of a 0.6 m thick composite layer of granular and geotextile to be constructed above the existing cells to act as a hydraulic control layer for leachate collection, the disposal capacity is reduced to about 3.6 million m^3.

The waste disposal capacity for Alternative Method 2 is required to be generally similar, and the size of the disposal area and the top of waste contours, have been adjusted accordingly. The resulting total air space is estimated as 3.75 million m^3, which allowing for internal sub-cell separation berms, is reduced to about 3.6 million m^3.

2.2.1 Alternative Method 1 - Vertical Expansion of Existing Landfill Site

The vertical expansion alternative will be constructed within the footprint of the existing landfill (**Figure 1**). The proposed area to be landfilled as indicated in the CDR covers 55.6 hectares (CRA, 2014b). The conceptual design for Alternative Method 1 is described in the CDR and illustrated in CDR Figures 4 through 8, and Figures 15 through 17 (CRA, 2014b).

2.2.1.1 Key Landfill Design Features

Alternative Method 1 involves a significant departure from the current more passive approach to landfill design. To minimize leachate mounding within the landfilled area, the design will incorporate an engineered drainage layer placed over the existing landfill cap. This drainage layer will be tied into a hydraulic control trench located around the perimeter of the landfill.

To reduce infiltration and the volume of leachate produced, an engineered cover system will be systematically placed over the areas of the landfill site as they are brought to final grade and closed. These features are described in greater detail herein.

Hydraulic Control System*:* A hydraulic control system, consisting of a drainage layer/blanket below the new waste that is tied into a perimeter hydraulic control trench, will be installed as landfilling progresses. The hydraulic control layer is intended for leachate control. The location of the perimeter trench with manholes is shown in CDR Figure 4. Schematic profiles of the collection system to be constructed in different areas of the site are included in CDR Figures 15 through 17 (CRA, 2014b).

As proposed, the hydraulic control system will include the following elements:

- A perimeter trench installed to an approximate depth of 5 m around the landfill site and backfilled with drainage stone with high igneous and metamorphic rock content. The 5 m depth of the perimeter trench has been selected to cut-off movement of shallow groundwater/leachate from the landfill through the shallow weathered zone of the native overburden.

  [Note: A shallower (1 m to 2 m deep) ‘interim’ collection system will be installed between Manholes 1 and 3 (distance of 388 m) near the Central Processing Area of the Facility and the Pre-1986 Landfill, because various structures encroach on the landfill in this area. On closure of the overall Lambton Facility, the need to install a deeper hydraulic control trench will be reassessed. If the results of groundwater monitoring indicate shallow migration is occurring beyond design expectations, temporary measures to control leachate movement could be installed as a ‘contingency’ west of the Central Processing Area.]

- The perimeter trench will be drained to 9 m deep manholes installed at regular distances along the trench. The manholes will be equipped with float activated pumps that discharge to a forcemain, which directs flow to a storage tank(s). The trigger levels for individual sumps that control segments of the trench could be individually adjustable, based on the hydraulic head in the waste and adjacent water sources (invert level of ditches and potentiometric surface in the Interface Aquifer).

- A continuous granular drainage layer installed below the waste. The layer will consist of 0.6 m of drainage stone placed over a woven geotextile and covered by a nonwoven geotextile. This internal drainage layer is intended to capture leachate from two sources:

  1) leachate ‘squeezed’ from the existing waste as a result of the weight of the added new waste and compression/consolidation of the existing waste; and

  2) leachate generated by gravity drainage from the new waste placed above the drainage layer.

  During construction of the hydraulic control layer, the residual cap over the existing landfill will be penetrated by shallow excavations into the underlying waste with the excavations backfilled with stone. This is to provide hydraulic connection with the waste to relieve the hydraulic head pressure in the existing waste that is expected to form as a result of the build-over.

  The granular drainage layer will extend out to, and will be hydraulically connected with the perimeter collection system.

The collected leachate will be treated on-site (at the Facility incinerator) or will be sent for treatment off-site.

Landfill Cap: An engineered cap will be placed over the completed areas of the site, as development proceeds. As described in the CDR (CRA, 2014b), the cap from the vegetated surface down to the top of the waste will consist of:

15 cm thick topsoil layer
35 cm thick protective layer
20 cm thick sand layer
0.15 cm HDPE Geomembrane
0.6 cm thick Geosynthetic clay liner
15 cm thick sand layer
60 cm thick interim clay cover
Top of Waste

Construction will involve the initial placement of an interim cap over the waste with the final cover installed progressively. The final cover will be extended outward from the landfill over the perimeter leachate collection trench and a clay plug (approximately 5 m width by 5 m depth) to be installed external to the trench. The HDPE geomembrane will be anchored in a 3 m deep trench, just beyond the clay plug. The clay plug and the HDPE geomembrane in the anchor trench are intended to reduce the volume of lateral groundwater movement through the upper weathered overburden inwards towards the trench. A schematic profile of the cover showing the anchor trench is included in CDR Figure 17 (CRA, 2014b).

Surface Water Management: The existing surface water management system at the landfill will be altered to accommodate construction of Alternative Method 1. The stormwater management criteria that have been specified in the CDR are:

- drainage ditches designed to accommodate runoff of 1:25-year rain-fall event;
- detention capacity on site to accommodate runoff for a 1:100-year storm; and
- minimum surface water treatment capacity of 2 million litres per day.

Proposed alterations to the surface water management system are shown in CDR Figure 7 (CRA, 2014b).

The alterations to the surface water drainage will involve:

- closure of the existing west surface water pond and construction of a new pond closer to the west property boundary;

- a reduction in the size of the east surface water pond; construction of a new ditch between the landfill and the perimeter berm at the east side of the property;

- regrading of the northern internal ditch to divert flow both to the west and south around the existing landfill from the northeast corner of the site; and

- excavation of new drainage channel to divert flow southward along the west side of the Central Process Area.

The drainage ditches internal to the Lambton Facility property will be widened to provide additional retention capacity.

The current pumping from the east pond to 'lift' water to the drainage ditch south of the Pre-1986 Landfill Area will be continued, with a pump house constructed for the equipment.

The surface water will be treated as necessary to maintain compliance with applicable regulatory policies prior to release to the east ditch along Telfer Road. Treatment will occur at the Lambton Facility Water Treatment Plant located immediately downstream of the west retention pond. The Water Treatment Plant will have the capacity to be operated at a rate of 2 million litres per day or greater, as currently approved.

Process Water: Runoff from the Central Process Area, roadways and the container laydown areas has the potential to be impacted and will be managed separately. Specifically, the water will be contained, diverted to existing process water retention ponds via drainage swales or in isolated locations by pumping, and disposed of the process water at the Facility incinerator.

The locations of the existing process water ponds are shown in CDR Figure 4. The existing process water retention pond located south of the Facility incinerator will be filled and replaced with a new process water pond located to the southwest [CDR Figure 2 (CRA, 2014b)].

2.2.1.2 Assumptions that Impact Geology/Hydrogeology Criteria

The design will incorporate engineered features that complement the passive structures and operational practices that are currently in place to mitigate the effects of the existing landfill on groundwater. These engineered features include a composite cover system that will minimize infiltration and a hydraulic control layer and hydraulic control trench (HCT) or a perimeter leachate collection trench (LCS). It is assumed that the engineered cover and the perimeter hydraulic control trench will be accessible and will be repaired or replaced, as necessary, in-perpetuity in the future.

There will be ongoing requirements for:

- monitoring of surface water and ground water quality, shallow and deep groundwater levels, surface water discharge volumes; and
- management (control and treatment) of surface water and groundwater discharge from the Facility property.

Figure 4, Figure 5, Figure 6 and Figure 7 are conceptual cross sections showing the design features for the Alternative Method 1 vertical construction over various areas of the Existing Landfill (i.e., Pre-1986 Landfill, Pre-1986 Landfill between Manhole 1 and Manhole 3, Cells 16/17 and Cell 18). The cross sections are representative of physical conditions at the cross section locations shown in **Figure 1**.

The construction of the internal hydraulic control layer and perimeter collection trench (LCS) will reduce the leachate head within the landfill and provide for the collection of leachate moving outward from the waste in the shallow subsurface to the perimeter of the landfill area. The influence of the landfill on groundwater quality is limited to contaminant discharge outward from the sidewalls and base of the landfill.

Over the long-term, this movement is dominated by advective transport and diffusion is an insignificant component of mass transport. As solutes migrate from the landfill, the concentration gradients will decline through time and diffusion will become a progressively less significant component of mass transport. Conservative constituents such as chloride, which are not readily degraded or attenuated, will in time reach the underlying Interface Aquifer.

The analysis of net effects applies equations presented in Section 2.1 to estimate discharge volumes and chloride loadings.

The assessment of the Pre-1986 Landfill Build Over (Figure 4 and Figure 5), Cells 16/17 Build Over (Figure 6), Cell 18, Sub-cells 4-12, and 14 Build Over (Figure 7), Cell 18, Sub-cell 1 and 2 (Figure 8), and Cell 18 Sub-cell 15 (Figure 9) focuses on three pathways, namely shallow flow with discharge to the perimeter LCS (referred to in Figures 4 through 9 as QLCS) and deep flow with discharge to the Interface Aquifer (referred to as QD and QB). As illustrated in Figure 5 (Pre-1986 Landfill Build-Over between Manhole 1 and Manhole 3), the perimeter LCS will only extend to a maximum depth of 1 to 2 m, because of the large number of structures adjacent to the landfill.

Germaine to this analysis is a set of 'parameters' unique to Alternative Method 1 that are derived/developed from the existing conditions at the Facility property and assumptions about future conditions. The landfill dimensions (areas and perimeter lengths) are shown in Figure 1. The engineered features and physical setting (geology, overburden thickness, hydraulic conductivity of the major units and projected water levels) are conceptualized in Figures 4 through 9. Parameters considered in the analysis are listed in **Table 2** and their derivation is briefly discussed below:

Infiltration Rate: The potential long-term leachate generation rate for the cover system was estimated by CRA (2014b) using the Hydrologic Evaluation of Landfill Performance (HELP) model version 3.07 developed by the Environmental Laboratory, US Army Corps of Engineers (Schroeder et al., 1994). The HELP model considers a number of factors including vegetation cover, layer thickness and hydraulic conductivity, climate, evaporation, evapotranspiration and runoff. Percolation/leakage through the layers is primarily controlled by hydraulic conductivity. The annual rate of infiltration through the proposed engineered cover is estimated at about 3 mm per year. The calculations applied to Alternative Method 1 and the proposed cover to be installed over the Pre-1986 Landfill for the future baseline condition, are based on the assumption that this leakage is correct.

Leachate Level in Landfill: Per CDR Figure 17, the hydraulic control layer to be installed internal to the landfill, blanketing the existing waste, will be tied into a perimeter collection system. Along most of the perimeter of the landfill, the LCS will be installed to a depth of 5 m (invert elevation between 195 mASL and 196 mASL). For a length of about 388 m between Manhole 1 and Manhole 3 (Figure 1 and Figure 5), the depth of the LCS will be limited to 1 m to 2 m (199 - 200 mASL).

The stabilized leachate level in the landfill over the long-term depends on the infiltration rate through the engineered cover, and the outward seepage from the sides and base of the landfill excavation. A balance is expected to be achieved where infiltration volume is equal to the volume of outward seepage.

Table 2. Parameters Considered in the Effects Analysis for Alternative Method 1

Parameter	Pre-1986 Landfill	Cell 16 &17	Cell 18 (Sub-cells 4-12 & 14)	Cell 18 (Sub-cell 15)
Landfill Dimensions				
Footprint of landfill at top of waste	267,345 m^2	192,139 m^2	90,484 m^2	40,950 m^2
Exterior Perimeter of landfill at top of waste	Full perimeter - 1,525 m (between Manhole 1 to 3 - 388 m)	1,106 m	1,265 m	506 m
Landfill Elevation on closure	201 – 213 mASL	201 – 213 mASL	201 – 211 mASL	201 -210 mASL
Elevation at Toe of Landfill	200 – 202 mASL	201 mASL	201 mASL	201.5 mASL
Depth of excavation below grade	7.6 - 18.3 m	18.3 m		
Engineered Landfill Cover and Perimeter Leachate Collection System				
Engineered Landfill Cover System	Approx. 1.46 m thick. Per CDR (CRA, 2014b).			
Invert depth (elevation) of LCS	5 mBGS (elevation 195 - 196 mASL); 1-2 mBGS (199 -200 mASL between Manhole 1 and 3 (388 m length).			
Target operating Level of LCS	Liquid elevation of 198 mASL, except between Manhole 1 and 3, where limited by depth of LCS to 200 mASL.			
Separation distance between waste & LCS	2 m from CDR Figure 15 (CRA, 2014b).			
Leachate Elevation in Waste Cells				
Measured in Monitoring Wells (2013)	201.8 - 206.2 mASL.	196.4 – 204.4 mASL	198.7 – 199.4 mASL	196.5 – 199.5 mASL
Projected Level on Closure	Level projected to decline in future following installation of engineered cover system, hydraulic control layer and perimeter leachate collection trench (LSC). Based on a water balance, future steady state leachate elevation estimated as 198.12 mASL.			
Surface Water				
Invert elevation in ditch	198 - 201 mASL	200 mASL	200.5 mASL	201 mASL
Separation distance waste to ditches	10 m from CDR Figure 15 (CRA, 2014b).			
Groundwater Level Elevation				
Water Table	Varies by location (topography) and season, between 201 to 195 mASL, average 198 mASL.			
Potentiometric surface Interface Aquifer	Varies by location and season, between 201.5 to 195 mASL. Levels expected to continue to rise below northern portion of property. Value of 198 mASL applied in the analysis to remain conservative.			
Geology				
Hydraulic Conductivity of Clay Overburden	Weathered clay to 3 m (2 x 10^{-7} m/s); mildly fractured clay to 6 m (4 x 10^{-9} m/s); and intact clay below 6 m (2.6 x 10^{-10} m/s).			
Average Interface Aquifer Elevation	160 mASL	159 mASL	155 mASL	161 mASL
Effective Porosity of Clay Overburden	0.34			
Chloride Diffusion Coefficient	4 x 10^{-6} cm^2/s (4 x 10^{-10} m^2/s).			
Chloride Concentration in Leachate	38,100 mg/L (average value for recent waste placed in Cell 18).			

The primary variable in this water balance is the value assigned to the operating water level of the LCS. As the LCS and waste will be hydraulically connected, adjustment of the operating level of the LCS will result in the level in the waste declining. With the lower leachate head the hydraulic gradients between the waste and the LCS, perimeter ditches and the Interface Aquifer will decline and the amount of flow moving towards these receptors will be altered.

Analyses were developed to investigate the implications of altering the leachate collection system design parameters (depth and water level) on the potential performance of Alternative Method 1 (see SSPA memorandum provided in Appendix D.2).

The results of the analyses indicate that containment of the waste would occur as long as the water level in the LCS is maintained below the long-term average water levels in the Clay Aquitard and the Interface Aquifer.

The base analysis applied in this assessment was a long-term operating level in the LCS of 198 mASL. Lowering the operating level in the LCS would induce an increasing volume of lateral flow towards the LCS from areas beyond the landfill perimeter and would also result in an upward hydraulic gradient from the Interface Aquifer and a contribution of flow from this water source to the LCS.

The operating level of the LCS for individual segments of the perimeter trench will need to be determined in the future based on the monitored hydraulic response in the adjacent vicinity. The level will likely be set within a target range dictated by the depth (i.e., invert level of the LCS) and an upper bound determined by the depth of adjacent drainage ditches and the potentiometric level in the Interface Aquifer. The goal will be to maintain a hydraulic gradient that is inward to the LCS from the waste, and generally flat to slightly inward from adjacent water sources (surface ditches and Interface Aquifer).

For purposes of the screening level assessment, the stabilized leachate level in the landfill over the long-term has been estimated by applying a water balance approach, which considers the predicted infiltration rate through the cap and the outward seepage from the sides and base of the landfill excavation. A balance is achieved where infiltration volume is equal to the volume of outward seepage.

The analysis involved a manual adjustment of the leachate level assigned to the landfill (above the level set at the LCS) in a spreadsheet until the seepage outflow through the sides and base of the landfill balanced the infiltration rate of 3 mm/year through the engineered landfill cover. The spreadsheet (Table A.1-1) is included in Appendix A.1.

Hydraulic Conductivity: The calculation of Darcy flux from the landfill requires an estimate of the hydraulic conductivity of the fill/overburden adjacent to the waste. The hydraulic conductivity values applied in the net effects analysis are presented in Figures 4 through 9. These values were compiled from a combination of testing results for fill and native overburden, and prescriptive design requirements for clay compaction presented in the Facility D & O.

As described in the Geology and Hydrogeology Existing Conditions Report (RWDI, 2014e), the Lambton Facility is underlain by a significant thickness of overburden that is dominated by clay-rich till and lacustrine deposits. The upper portion of the clay-rich sediment is influenced by atmospheric weathering and is characterized by fracturing and chemical oxidation producing a blocky structure.

The open fractures in the weathered zone are conducive to groundwater movement and allow for the rapid dispersion of infiltrating precipitation. The hydraulic conductivity of the clay overburden is influenced by the intensity of fracturing and decreases with depth as the intensity decreases.

The layer of weathered/fractured clay overburden at surface (referred to as the Active Aquifer) was subdivided into two portions based on the intensity of fracturing and the expected hydraulic activity. This includes an upper portion with a thickness of 3 m and a lower portion with a thickness of 2 m. The division into two units and the assigned thicknesses were subjective, being based on numerous references (e.g., Ruland et al., 1991; Figure 2) that indicate the intensity of weathering and fracturing decreases significantly below about 3 m. Groundwater flow is expected to be preferentially through the fractures and it is reasonable to conclude that the volume of lateral flow would similarly decrease below 3 m. This is evident in various profiles of bromide concentrations that are included in McKay, Gillham and Cherry (1993). Bromide is a commonly applied tracer that is not readily attenuated and would be expected to move with groundwater. As illustrated in this reference, bromide concentrations are attenuated below 3 m, indicating the bulk of the injected bromide is moving with groundwater above this depth.

The full depth of the Active Aquitard is uncertain [i.e., comments received from the PRT on the draft Net Effects Analysis & Comparative Evaluation Report (January 2014)]. Fractures are known to extend to depths in excess of 10 m based on indirect evidence (e.g., detectable tritium) but the general consensus in the references cited in the Existing Conditions Report (RWDI 2014e) is that the degree of hydraulic activity declines significantly below about 5 m. This is attributed to the presence of few if any horizontal fractures that would convey flow laterally. To remain conservative in the net effects analysis, the thickness of the Active Aquitard has been increased from 5 m to 6 m. The Active Aquitard pathway has therefore been subdivided into upper and lower components each 3 m thick. The various figures in this report and the summary tables have been adjusted accordingly.

Values of hydraulic conductivity considered in the analysis and cited references are summarized below:

- upper portion of the weathered zone with extensive fracturing to a depth of about 3 m (2×10^{-7} m/s), bulk averaged value from seepage influx in a 5.5 m collector trench (McKay, 1991);

- lower portion of the weathered zone with less intensive fracturing between 3 - 6 m (3.66×10^{-9} m/s rounded to 4×10^{-9} m/s) from compilation of hydraulic conductivity values for single response testing conducted since 1991 (RWDI 2014e);

- unweathered/intact clay below 6 m (2.6×10^{-10} m/s), consolidation testing (McKay, 1991); and,

- compacted clay used in the cap and placed in the 'key' cut into the sidewall of the landfill excavation (1×10^{-9} m/s) as prescribed in the design requirement in 2009 Design and Operations Report for Cell 18 & LDR Pretreatment-Lambton Facility (Clean Harbors Canada, Inc., 2010), which is commonly referred to as the D&O Report.

With regards to the Alternative Method 1 'build-over', an engineered cover will be constructed over the landfill as individual areas of the landfill reach final grade. The addition of the cover system will minimize disturbance/weathering of the existing clay cap/clay key that underlies the cover.

As noted above, the hydraulic conductivity of this material (1×10^{-9} m/s) is prescribed in the design requirement in *2009 Design and Operations Report for Cell 18 & LDR Pretreatment-Lambton Facility* (Clean Harbors Canada, Inc., 2010). However it is reasonable to conclude that the installation of a hydraulic control trench (i.e., LCS) along the perimeter 2 m away from the waste, will resultant in pressure relief fracturing of the residual clay cap/key. This will alter the hydraulic conductivity of the clay. It is therefore assumed that a 100-fold increase in the hydraulic conductivity to 1×10^{-7} m/s is possible. This higher value for hydraulic conductivity will be applied in the estimation of seepage to the perimeter LCS for Alternative Method 1.

Effective Porosity: The calculation of chloride loading by the process of diffusion requires a value for effective porosity of the clay and an effective diffusion coefficient for chloride movement through the clay. Field and laboratory studies at the site have shown that the effective porosity varies over a relatively narrow range. Desaulniers (1986) reports a porosity of 0.40. Bedard (1990) reported a porosity of 0.38. Myrand et al. (1992) reported a mean porosity of 0.34 from 10 core samples of unweathered/intact till at ~4.5 m to 5.0 m depth.

This value is similar to the values reported in McKay et al. (1993) and in Johnson et al. (1989) that were assumed in their analyses of in-situ diffusion profiles at the Lambton Facility. For purposes of the analysis an effective porosity of 0.34 is assigned to both the clay overburden and the clay cap (where applicable).

Diffusion Coefficient for Chloride in Clay: A number of chloride concentration profiles have been developed through the cap overlying the waste at the Lambton Facility as part of the monitoring program that is in effect at the Facility. These profiles were computer simulated resulting in a 'best fit' where an effective diffusion coefficient of 4×10^{-6} cm^2/s (4×10^{-10} m^2/s) is specified for the clay. This value for the effective diffusion coefficient is applied in the diffusion analysis.

Chloride Concentration in Leachate: A field program was undertaken as part of the EA field investigations conducted to collect information on liquid levels and the composition of the leachate in the waste cells. For purposes of the chloride loading calculations it is assumed that the chloride concentration (38,100 mg/L) in waste in Cell 18, Sub-cell 5, is representative of waste currently received at the Lambton Facility and likely to be received in the future for disposal. This chloride concentration (38,100 mg/L) has been applied to both Alternative Methods and in the assessment of the future condition at Cell 18, Sub-cell 15, which is currently under construction.

2.2.2 Alternative Method 2 - Shallow Entombed Landfill South of Existing Site

Alternative Method 2 involves the construction of a new landfill on property owned by Clean Harbors Canada, Inc. located south of the existing landfill site (**Figure 2**). The landfill volume capacity is generally equivalent to the design capacity for Alternative Method 1 (about 3.75 million m^3). To achieve this capacity, the southern alternative will occupy an area of 38.7 hectares. The conceptual design for Alternative Method 2 is described in the CDR and illustrated in CDR Figure 9 through Figure 14, and Figure 18 (CRA, 2014b).

2.2.2.1 Key Landfill Design Features

The Alternative Method 2 design, per CDR Figures 9 and 10 (CRA, 2014b), involves the excavation of landfill cells to an elevation of 186.5 mASL (or 12.5 m below the existing ground surface).

3. Net Effects Analysis Results

3.1 Groundwater Quantity Criterion

The analysis involves the preparation of a surface water and groundwater water balance based on available climatic data (precipitation and evapotranspiration) and an estimate of volume of seepage/leakage from the Existing Landfill (baseline condition) and each of the proposed Alternative Methods. This estimate considers the Darcy Flux multiplied by the area across which seepage occurs.

The seepage calculations consider three distinctive pathways for the advective transport of leachate. These include:

- the shallow lateral movement of leachate through the clay cap and the hydraulically-active, weathered overburden with discharge occurring either at the perimeter LCS (referenced to as QLCS) or to perimeter drainage ditches (referenced as Q_S);

- deeper groundwater movement (referenced as Q_D) outward from the excavation slope with discharge to the Interface Aquifer [Note: the analysis in the previous draft report assigned this discharge to the drainage ditches. Based on the quasi-two dimensional analysis provided in the SSPA Memorandum in Appendix D.1, seepage discharge was reassigned to the Interface Aquifer.]; and

- movement downward from the base of the waste cells (referenced as Q_B) with discharge to the Interface Aquifer.

The seepage rates, per the discussion in Section 2.1, are influenced by the hydraulic conductivity of the material, the leachate levels in the waste cells and water level at the discharge receptor (i.e., perimeter LCS, perimeter drainage ditches and Interface Aquifer), the separation distance between the waste and the discharge receptor and the area across which movement occurs.

The calculations were completed by spreadsheet (Appendix A) with the results presented in a series of conceptual cross sections developed for the Existing Landfill (baseline condition) and each of the Alternative Methods, which are also included in Appendix A.

It is proposed that purge wells be used in the future to mitigate groundwater quality impact of the Interface Aquifer. During the EA, the MOE reviewer observed that the intended future use of purge wells would alter the hydraulic gradient adjacent to and below both the Existing Landfill, and below the two Alternative Methods under consideration. This would influence the net effect on groundwater flow and chloride mass discharge to surface water and the Interface Aquifer.

To address this comment a groundwater model was developed (SSPA Memorandum, Appendix D.3) and used to assess the effect of purge well pumping on hydraulic gradients below the Existing Landfill and the two Alternative Methods. The net effects analysis presented in this Report has been expanded to include the influence of the purge wells. The calculations, which incorporate the influence of the purge wells, were also completed by spreadsheet (Appendix B).

Table 11. Leachate Movement from Existing Landfill under Active Pumping

Landfill Area	Leachate Flow Paths			
	Discharge to Leachate Collection System (QLCS)	Lateral Discharge to Drainage Ditches (QS)	Downward Discharge to Interface Aquifer	
			From Excavation Sidewalls (QD)	From Excavation Base (QB)
Pre-1986 Landfill	610.0 m^3/year	Flow inward towards landfill	8.6 m^3/year	96.1 m^3/year
Cell 16 and Cell 17		1,356.0 m^3/year	32.8 m^3/year	234.0 m^3/year
Cell 18, Sub-cells 1 and 2		329.8 m^3/year	36.6 m^3/year	160.6 m^3/year
Cell 18, Sub-cell 3 Pumping from HCL	100.0 m^3/year	not applicable		
Cell 18, Sub-cells 4 – 12 and Sub-cell 14		2,669.0 m^3/year	30.9 m^3/year	318.0 m^3/year
Cell 18, Sub-cells 15*		1,302.1 m^3/year	5.4 m^3/year*	45.4 m^3/year*
Totals	**710.0 m^3/year**	**5,656.9 m^3/year**	**968.4 m^3/year**	

(*) Sub-Cell 15 constructed over Cells 16/18, incremental contribution from leachate mound that will develop.

An evaluation of purge well use at the Facility property is provided in SSPA Memorandum, Appendix D.3. It was determined that two purge wells pumping at a combined rate of 6 L/minute (3,152.6 m^3/year) would provide complete hydraulic containment in the Interface Aquifer below the Existing Landfill. The Existing Landfill occupies an area of 533,554 m^2 (53.3 hectares). The extraction rate per hectare is 59.1 m^3/year. The extraction rate for the purge wells (3,152.6 m^3/year) is approximately 3 times the rate of 968.4 m^3/year estimated for pathways QD and QB from the landfill. The balance of the water extracted would be drawn from beyond the perimeter of the landfill.

The total volume of groundwater extracted from the Interface Aquifer (3,152.6 m^3/year) will need to be managed as leachate in a manner similar to the water collected from the perimeter LCS and the water extracted from the HCL in Cell 18, Sub-cell 3.

3.1.2 Alternative Method 1

3.1.2.1 Vertical Expansion over Previously Landfilled Areas

The water balance for Alternative Method 1 considers the construction of a landfill with engineered features (cover system and leachate collection) within the footprint of the existing landfill.

Figures A.2-1 through A.2-4 (Appendix A), are conceptual cross sections through portions of the proposed Alternative Method 1 landfill that will be constructed above the Pre-1986 Landfill, Cells 16 and Cell 17, and Cell 18. See Figure 1 for the locations and dimensions.

The engineered cover system has an inherently low hydraulic conductivity, which would significantly reduce the volume of infiltration. The proposed leachate collection system includes a hydraulic control layer (or granular blanket) to be placed at the base of the cell below the new waste and will be tied into a perimeter trench.

The hydraulic control layer and collection system are important to the initial displacement of leachate present in the older underlying cells and any leachate generated while the new cells are open. It is expected that the effectiveness of the hydraulic layer will decline with time due to the potential for the granular layer to plug. The perimeter LCS is accessible and can be repaired and/or replaced so any leachate moving outward along the perimeter of the landfill through the shallow subsurface will continue to be collected. This will effectively eliminate shallow discharge to the perimeter ditches. Deeper lateral flow through the intact clay will however continue. This discharge is assigned to the Interface Aquifer.

The water balance for Alternative Method 1 is presented in Table A.2-1, Appendix A and summarized in **Table 12**. The methodology employed for calculating the water balance for Alternative Method 1 is described in Section 2.2.1.2. A leachate head of 198.12 mASL was applied to balance infiltration and outward seepage.

Table 12. Leachate Movement from Alternative Method 1 Landfill via Pathways

Landfill Area	Leachate Flow Paths			
	Discharge to Leachate Collection System (QLCS)	Lateral Discharge to Drainage Ditches (QS)	Downward Discharge to Interface Aquifer	
			From Excavation Sidewalls (QD)	From Excavation Base (QB)
Pre-1986 Landfill (Build over)	745.5 m^3/year		0.7 m^3/year	12.1 m^3/year
Cells16 and 17 (Build over)	597.9 m^3/year		0.4 m^3/year	8.0 m^3/year
Cell 18 Sub-cells 4 – 12, 14 (Build Over)	205.2 m^3/year		0.5 m^3/year	3.2 m^3/year
Cell 18 Sub-cell 15 (Build Over)	52.1 m^3/year		0.2 m^3/year	1.8 m^3/year
Totals	**1,600.7 m^3/year**	**not expected**	**26.9 m^3/year**	

Active Pumping from Interface Aquifer: Purge wells are to be employed in the future to extract the impacted groundwater from the Interface Aquifer. Pumping of the purge wells will induce a drawdown cone that extends outward from the locations of the wells (see SSPA Memorandum, Appendix D.3) and will alter the hydraulic gradients below the Facility property.

The leachate head for the various cells was adjusted to 198.09 mASL from 198.12 mASL to balance the increase in the discharge to the Interface Aquifer with the assigned leakage (3 mm/year) through the engineered cover. The adjustment in the leachate head results in a decrease in the discharge to the perimeter LCS from 1,587.4 m^3/year to 1,161.5 m^3/year. The discharge to the Interface Aquifer will increase from 91.2 m^3/day to 484.0 m^3/day.

The water balance under active pumping from the Interface Aquifer is presented in Table B.2-1, Appendix B and summarized in **Table 13.**

Table 13. Leachate Movement from Alternative Method 1 under Active Pumping

Landfill Area	Leachate Flow Paths			
	Discharge to Leachate Collection System (QLCS)	Lateral Discharge to Drainage Ditches (QS)	Downward Discharge to Interface Aquifer	
			From Excavation Sidewalls (QD)	From Excavation Base (QB)
Pre-1986 Landfill (Build over)	528.1 m^3/year		6.6 m^3/year	109.6 m^3/year
Cells16 and 17 (Build over)	423.5 m^3/year		10.8 m^3/year	191.8 m^3/year
Cell 18 Sub-cells 4 – 12, 14 (Build Over)	145.3 m^3/year		18.4 m^3/year	130.8 m^3/year
Cell 18 Sub-cell 15 (Build Over)	36.9 m^3/year		7.4 m^3/year	61.8 m^3/year
Totals	**1,133.8 m^3/year**	**not expected**	**537.2 m^3/year**	

3.1.2.2 Combined Effect for the Existing Landfill and Alternative Method 1

In assessing the net effects of Alternative Method 1 it is necessary to include those portions of the Existing Landfill that will not be incorporated into the vertical expansion. These include the flow contribution from Cell 18, Sub-cells 1 and 2, and the water projected to be extracted from the Hydraulic Control Layer in Cell 18 Sub-cell 3.

The CDR (CRA, 2014b) design shows the perimeter LCS as extending around Cell 18 Sub-cells 1 and 2; therefore the LCS is expected to capture the shallow discharge from these sub-cells. The water balance for Cell 18 Sub-cells 1 and 2 is presented in Table A.2-2. The discharge rate for Cell 18 Sub-cells 1 and 2 is based on an operating level for the LCS of 198 mASL and an assumed constant leachate head in the sub-cell of 201 mASL. This would represent a conservative over-estimation of the actual seepage from the sub-cells, as the installation of the LCS will have the effect of lowering the leachate head in the sub-cells. The combined net effects for the Alternative Method 1 and portions of the Existing Landfill that will not be developed are summarized in **Table 14**.

Table 14. Leachate Movement from Alternative Method 1 and Existing Landfill

Landfill Area	Leachate Flow Paths			
	Discharge to Leachate Collection System (QLCS)	Lateral Discharge to Drainage Ditches (QS)	Downward Discharge to Interface Aquifer	
			From Excavation Sidewalls (QD)	From Excavation Base (QB)
Alternative Method 1 (from Table 12)	1,600.7 m^3/year		1.8 m^3/year	25.1 m^3/year
Cell 18, Sub-cells 1 & 2 (from Table A.1-1, Appendix A)	2,022.6 m^3/year		14.1 m^3/year	61.8 m^3/year
Cell 18, Sub-cell 3 Pumping from HCL	100.0 m^3/year	Not applicable		
Totals	**3,723.3 m^3/year**	**not expected**	**102.8 m^3/year**	

Active Pumping from Interface Aquifer: The increased hydraulic gradient associated with purge well pumping will result in an increase in the total volume of water that will need to be managed as leachate. The combined net effects for the Alternative Method 1 and portions of the Existing Landfill that will not be developed, under purge well pumping, are summarized in **Table 15**.

Table 15. Leachate Movement from Alternative Method 1 and Existing Landfill under Active Pumping

Landfill Area	Leachate Flow Paths			
	Discharge to Leachate Collection System (QLCS)	Lateral Discharge to Drainage Ditches (QS)	Downward Discharge to Interface Aquifer	
			From Excavation Sidewalls (QD)	From Excavation Base (QB)
Alternative Method 1 (from Table 13)	1,133.8 m^3/year		43.2 m^3/year	494.0 m^3/year
Cell 18, Sub-cells 1 & 2 (from Table B.2-2, Appendix B)	2,022.6 m^3/year		36.6 m^3/year	160.6 m^3/year
Cell 18, Sub-cell 3 Pumping from HCL	100.0 m^3/year	Not applicable		
Totals	**3,256.4 m^3/year**	**not expected**	**734.4 m^3/year**	

Preliminary modeling (Appendix D.3) identified that containment of groundwater in the Interface Aquifer for the Existing Landfill could be achieved by the use of purge wells pumping at a rate of 6 litres/minute (3,152.6 m^3/year). Alternative Method 1 occupies a slightly larger footprint (59 hectares) than the Existing Landfill, which based on an extraction rate of 59.1 m^3/year per hectare, results in total extraction volume of 3,486.9 m^3/year).

The purge volume exceeds the downward discharge to the Interface Aquifer by 2,752.5 m^3/year (3,486.9 m^3/year minus 734.4 m^3/year). This difference is groundwater drawn from beyond the perimeter of the landfill.

The leachate management system for Alternative 1 must therefore be capable of handling the volume of water that is: discharged to the leachate collection system (3,256.4 m^3/year), the water that is extracted from the Cell 18 Sub-cell 3 HCLs (100 m^3/year), and the water that is purged from the Interface Aquifer (3,152.6 m^3/year). The total volume to be managed as leachate is 6,509.0 m^3/year.

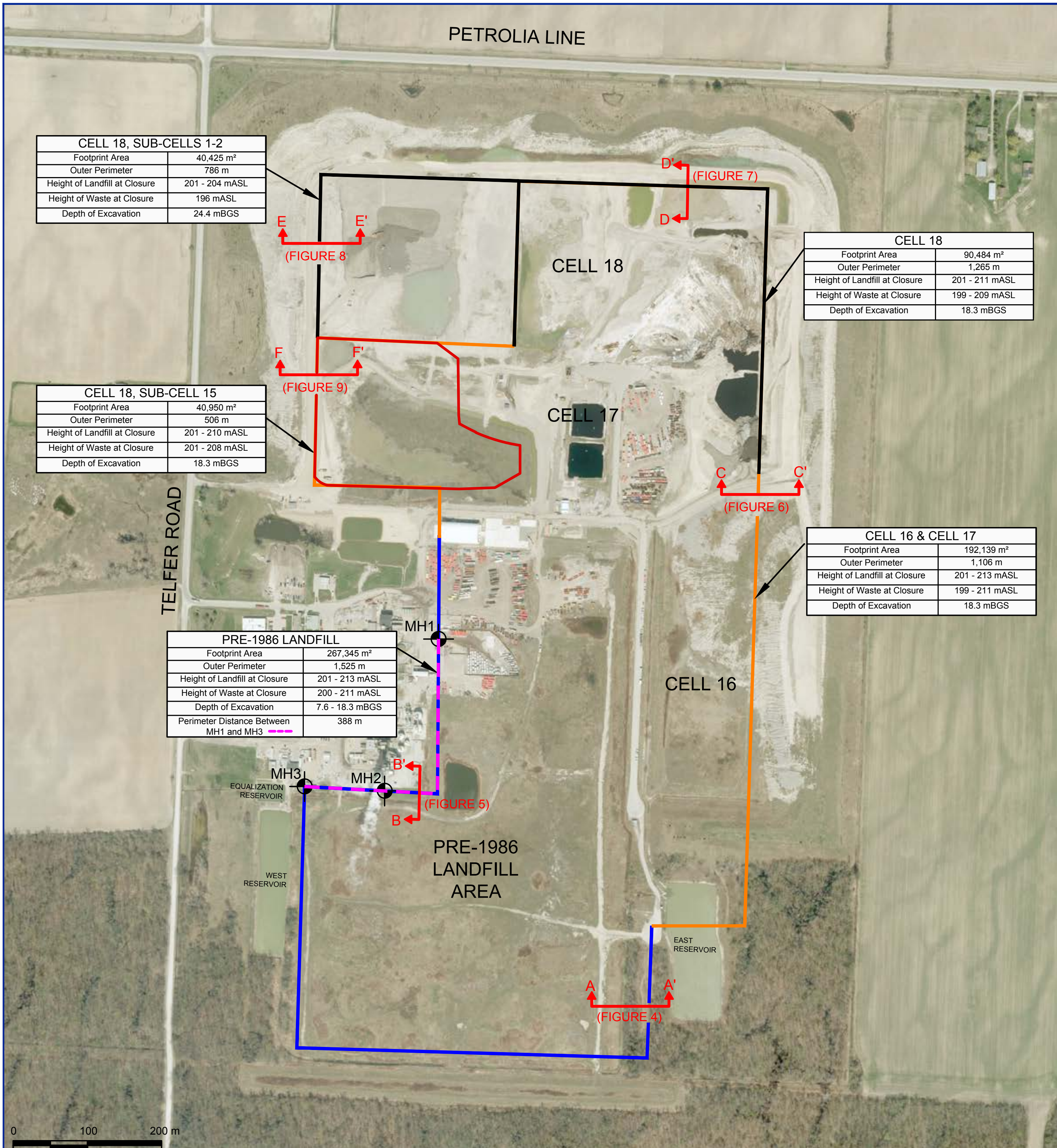
3.1.3 Alternative Method 2

3.1.3.1 New Landfill to South of Existing Landfill

Alternative Method 2 involves the construction of a new landfill on Clean Harbors' owned property south of the existing landfill site. The pathways for solute movement from the landfill are reflected in the conceptual cross section for this Alternative Method (Figure 8).

The discharge for this Alternative Method was determined for each pathway, namely shallow movement through the cap/key and Active Aquitard, and deeper lateral and downward movement from the landfill through the unweathered/intact clay to the Interface Aquifer. The approach is consistent with what was applied for Method 1.

Figures



LEGEND

- ALTERNATIVE METHOD 1 - PRE-86 LANDFILL OUTER PERIMETER
- ALTERNATIVE METHOD 1 - CELLS 16/17 OUTER PERIMETER
- ALTERNATIVE METHOD 1 - CELL 18 OUTER PERIMETER
- ALTERNATIVE METHOD 1 - CELL 18, SUB-CELL 15 OUTER PERIMETER
- A A' CROSS SECTION

NOTES:
1. ORTHOPHOTO PROVIDED BY FIRST BASE SOLUTIONS (2010).
2. AREAS INTERPRETED FROM FIGURE 1 FROM MOE PROJECT 044985 (DEC. 2013).

ALTERNATIVE METHOD 1
LOCATION AND DIMENSIONS

CLEAN HARBORS CANADA, INC.,
LAMBTON FACILITY LANDFILL

True North

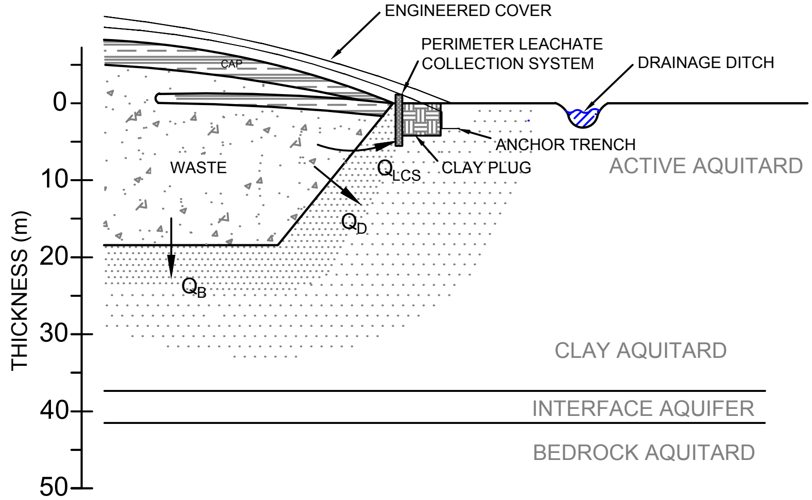
Project #1401210

Drawn by: SSL | Fig: **1**

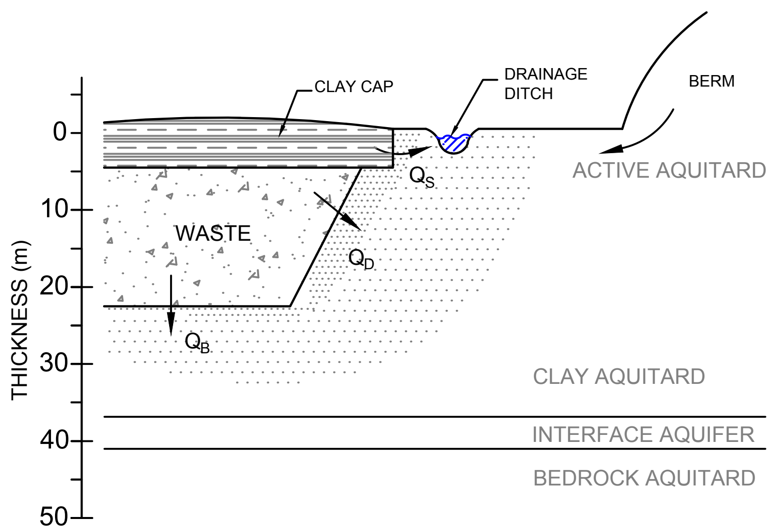
Approx. Scale: 1:7,000

Date Revised: June 23, 2014

PRE-1986 LANDFILL (WITH ENGINEERED COVER AND
PERIMETER LEACHATE COLLECTION SYSTEM AT CLOSURE)



POST-1986 LANDFILL
(SHALLOW ENTOMBED LANDFILL AT CLOSURE)



LEGEND

Q_{LCS} DISCHARGE TO LEACHATE COLLECTION SYSTEM
Q_S DISCHARGE TO SHALLOW GROUNDWATER
Q_D DISCHARGE TO DEEP GROUNDWATER
Q_B DISCHARGE TO BEDROCK

NOTES:
1. FIGURE MODIFIED FROM GROUNDWATER CONDITIONS AT LAIDLAW ENVIRONMENTAL SERVICES LAMBTON FACILITY BY JAGGER-HIMS LTD. (1996).

POTENTIAL PATHWAYS FOR CONTAMINANT MOVEMENT OUTWARD FROM EXISTING LANDFILL

CLEAN HARBORS CANADA, INC., LAMBTON FACILITY LANDFILL

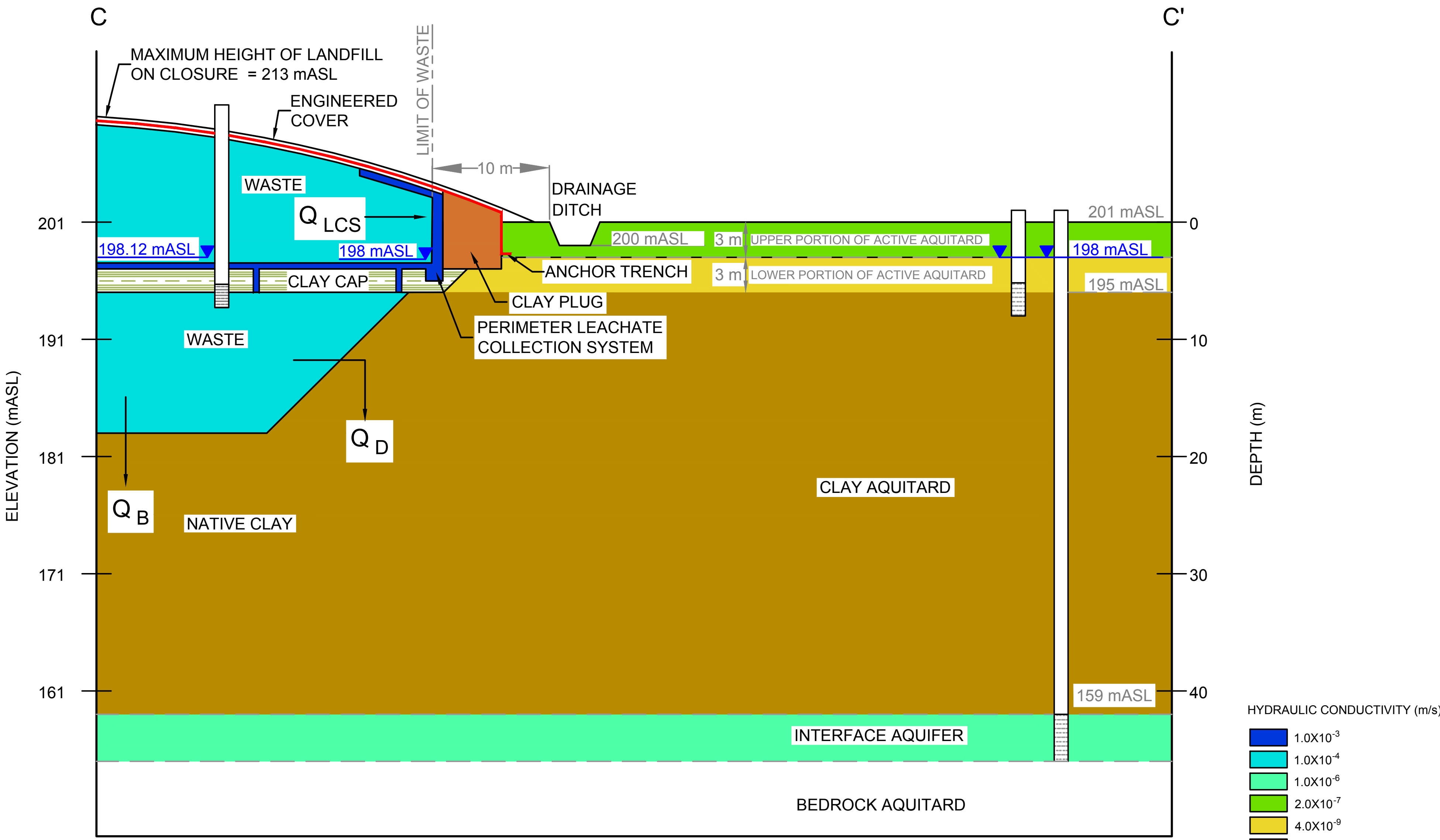
Project #1401210

Drawn by: SSL
Fig: 3

Approx. Scale: N.T.S.

Date Revised: June 12, 2014

RWDI



HYDRAULIC CONDUCTIVITY (m/s)

- $1.0X10^{-3}$
- $1.0X10^{-4}$
- $1.0X10^{-6}$
- $2.0X10^{-7}$
- $4.0X10^{-9}$
- $1.0X10^{-9}$
- $2.6X10^{-10}$
- $1.0X10^{-12}$

NOTES:
1. CROSS SECTION LOCATION C - C` REFERENCED IN FIGURE 1.
2. LANDFILL DIMENSIONS AND CONTAMINANT PATHWAY DISCUSSION PROVIDED IN SECTION 2.2.1 OF THE REPORT.

LEGEND

198.12 mASL	PROJECTED WATER LEVEL
Q_{LCS}	SHALLOW LATERAL MIGRATION TO THE PERIMETER LCS
Q_D	DEEPER MIGRATION TO INTERFACE AQUIFER
Q_B	VERTICAL MIGRATION TO THE INTERFACE AQUIFER

CONCEPTUAL CROSS SECTION ALTERNATIVE METHOD 1, CELLS 16 AND 17 BUILD OVER

CLEAN HARBORS CANADA, INC.
LAMBTON FACILITY LANDFILL

Drawn by: SSL
Fig: 6
Project No.: 1401210
Date Revised: June 12, 2014

RWDI

Clean Harbors Canada, Inc.
Lambton Landfill Expansion Environmental Assessment

Geology & Hydrogeology Net Effects Analysis & Comparative Evaluation Report

Volume II - Appendices

Prepared by:

CONSULTING ENGINEERS & SCIENTISTS

Tel: 289.205.0618
Fax: 519.823.1316

RWDI AIR Inc.
1100 South Service Road
Suite 324
Hamilton, Ontario, Canada
L8E 0C5
Email: solutions@rwdi.com

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October 2014

Appendices

Appendix A. Net Effects Analysis for the Existing Landfill (i.e., Baseline Condition) and the two Alternative Methods

Appendix B. Net Effects Analysis for the Existing Landfill (i.e., baseline condition) and the two Alternative Methods incorporating the use of Purge wells for Mitigation

Appendix C. Technical Memorandum Clean Harbors Lambton: Evaluation of Landfill Alternatives, S.S. Papadopulos & Associates, Inc., June 23, 2014.

Appendix D. Technical Memoranda prepared by S.S. Papadopulos & Associates, Inc. supporting aspects of the analysis.

Appendix D.1 Clean Harbors Lambton Facility: Net Effects analyses, Investigation of the implications of a quasi-two dimensional analysis approach, April 14, 2014.

Appendix D.2 Clean Harbors Lambton Facility proposed expansion: AM1 – Revised Sensitivity analyses for the leachate control system (LCS), May 14, 2014, revised October 8, 2014.

Appendix D.3 Clean Harbors Lambton Facility: Analysis of purge wells beneath the existing landfill, May 1, 2014 (revised May 27, 2014).

Appendix A

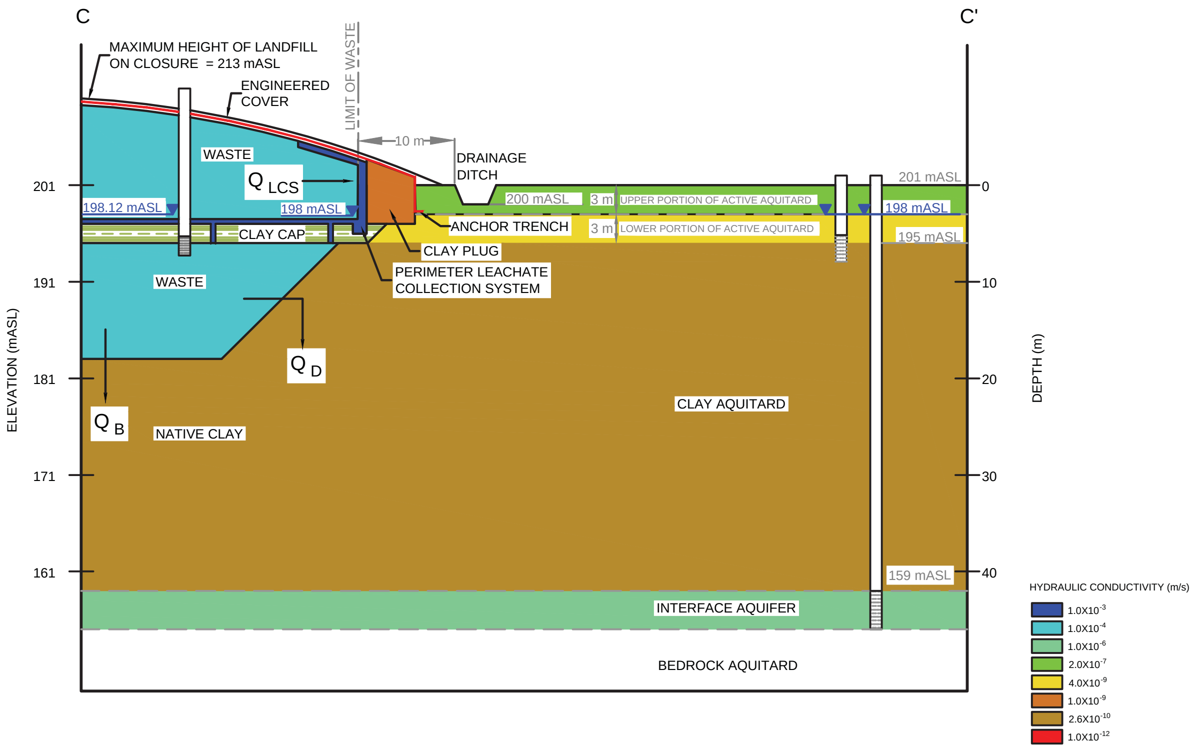
Net Effects Analysis for the Existing Landfill (i.e., Baseline Condition) and the two Alternative Methods

Appendix A.2 – Alternative Method 1

Table A.2-1. Water Balance Lambton Facility Landfill (Alternative Method 1)

		Alternative Method 1 (Vertical Landfill)															
		Constructed over Pre-1986 Landfill							Constructed over Cells 16 & 17			Constructed over Cell 18			Cell 18 (Sub-cell 15)		
		see Figure 5 (Cross Section B-B', Figure 1)			see Figure 4 (Cross Section A-A', Figure 1)				see Figure 6 (Cross Section C-C', Figure 1)			see Figure 7 (Cross Section D-D', Figure 1)			see Figure 9 (Cross Section F-F', Figure 1)		
	Units	Shallow Lateral Flow (QLCS and QS)															
Dimensions Considered in Analysis		Constructed over Pre-1986 Landfill**	QS via Active Flow Zone below LCS	QS via Less Active Portion of Active Aquitard	QLCS via Active Flow Zone	QLCS via Less Active Portion of Active Aquitard	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)
Ground Surface Elevation at Toe of Landfill	mASL	202.0	202.0	202.0	200.0	200.0	200.0	200.0	201.0	201.0	201.0	201.0	201.0	201.0	201.5	201.5	201.5
Landfill Properties																	
Height of Landfill on Closure (per Conceptual Design Report)	mASL	203 to 213	203 to 213	203 to 213	203 to 213	203 to 213	203 to 213	203 to 213	203 to 210	203 to 210	203 to 210	203 to 212	203 to 212	203 to 212	202 to 205		
Average Depth of Cell Excavation	m						18.3	18.3		18.3	18.3	18.3	18.3	18.3	18.3	18.3	18.3
Average Depth of Cell in Cap															4.0	4.0	4.0
Landfill Area (A) at Surface	m2							267,345.0			192,139.0			90,484.0			40,950.0
Exterior Perimeter Length of Landfill at Surface	m	388.0	388.0	388.0	1,137.0	1,137.0	1,525.0		1,106.0	1,106.0		1,265.0	1,265.0		506.0	506.0	
Active Aquitard																	
Total Active Aquitard Thickness	m	6.0	6.0	6.0	6.0	6.0	6.0		6.0	6.0		6.0	6.0		6.0	6.0	
Upper Portion of Active Aquitard (Active Flow Zone) Thickness	m	3.0	3.0	3.0	3.0	3.0									3.0		
Hydraulic Conductivity - Active Flow Zone	m/s	2.00E-07	2.00E-07		2.00E-07										2.00E-07		
Lower Portion of Active Aquitard (Less Active Zone) Thickness	m			3.0		3.0									3.0		
Hydraulic Conductivity - Lower Portion of Active Aquitard	m/s			4.00E-09		4.00E-09									4.00E-09		
Effective Hydraulic Conductivity Active Aquitard (where applicable)	m/s														1.02E-07		
Average Elevation of Interface Aquifer Below Landfill	mASL						160.0	160.0		159.0	159.0		155.0	155.0		161.0	161.0
Clay Aquitard																	
Clay Aquitard Thickness	m							40.0			42.0			46.0			40.5
Clay Aquitard Thickness Adjacent to Waste Cell	m						13.3			12.3			12.3			12.3	
Hydraulic Conductivity - Clay Aquitard	m/s						2.60E-10	2.60E-10		2.60E-10	2.60E-10		2.60E-10	2.60E-10		2.60E-10	2.60E-10
Engineered Features																	
Engineered Cover Thickness	m	2.0	2.0	2.0	2.0	2.0			2.0			2.0	2.0	2.0	2.0		
Width of Clay Plug beyond LCS	m	5.0	5.0	5.0	5.0	5.0			5.0			5.0			5.0		
Depth of Clay Plug	m	1.0	1.0	1.0	4.0	4.0			4.0			4.0			4.0		
Anchor Trench Depth	m	3.0	3.0	3.0	3.0	3.0			3.0			3.0			3.0		
Clay Cover Thickness	m	2.0	2.0	2.0	2.0	2.0			6.1			5.1			5.1		
Clay Key Perimeter Width	m								4.0			5.0			1.00E-09		
Hydraulic Conductivity of Weathered Clay Cover/Clay Key	m/s								1.00E-07			1.00E-07			5.1		
Depth of LCS	m	1.0	1.0	1.0	5.0	5.0			5			5			1.02E-07		
Invert Elevation of LCS	mASL	201.0	201.0	201.0	195.0	195.0			196.0			196.0			5.0		
															196.5		
Parameters Applied in the Discharge Calculation																	
Water Level Elevation (H, h), expressed in terms of elevation (mASL)																	
Assigned Leachate Level (H)	mASL																
Leachate Level in Waste Cell Estimated by Water Balance (H)	mASL	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12	198.12
LCS Operating Level (h)	mASL	201.0			198.0	198.0			198.0			198.0			198.0		
Ditch Invert (h)	mASL		201.0	201.0													
Interface Aquifer (h)	mASL						198.0	198.0		198.0	198.0		198.0	198.0		198.0	198.0
Distance (L) Along Flow Path to Receptor																	
Separation Distance Top of Waste to LCS	m	2.0	2.0	2.0	2.0	2.0	2.0		2.0	2.0		2.0	2.0				
Average Separation Distance Waste to LCS via Active Flow Zone	m	2.5			3.5				3.5			3.5			22.5		
Average Separation Distance Waste to LCS via Less Active Portion of Active Aquitard	m					6.5			4.5			4.5					
Separation Distance between LCS and Ditch	m		8.0	8.0	8.0	8.0	8.0		8.0	8.0		5.0	5.0				
Clay Cap / Key Width	m								3.5			4.0					
Separation Distance Top of Waste to Ditch	m		10.0	10.0			10.0		10.0	10.0		10.0	10.0				
Average Separation Distance Waste to Ditch via Less Active Portion of Active Aquitard	m		11.5												5.0		
Average Separation Distance Waste to Ditch via Active Aquitard	m			14.5								14.0			20.0		
Average Separation Distance Waste to Ditch via Clay Aquitard	m																
Separation Distance Waste to Interface Aquifer	m						27.4	21.7		29.9	23.7		33.9	27.7		28.4	22.2
Cross Sectional Area (A) for Groundwater Movement																	
Active Flow Zone (lateral flow)	m2	388.0			3,411.0												
Balance of Active Aquitard (lateral flow)	m2		776.0			3,411.0			5,530.0			7,590.0			3,036.0		
Clay Aquitard (lateral flow)	m2			1,164.0			20,282.5			13,603.8			15,559.5			6,223.8	
Landfill Area (vertical flow)	m2							267,345.0			192,139.0			90,484.0			40,950.0
Hydraulic Conductivity Applied along Flow Path	m/s	2.00E-07	2.00E-07	4.00E-09	2.00E-07	4.00E-09	2.60E-10	2.60E-10	1.00E-07	2.60E-10	2.60E-10	1.00E-07	2.60E-10	2.60E-10	1.0E-07	2.60E-10	2.60E-10
$Q = K (\Delta H/L) A$	m3/year	-2,819.2	-1,044.1	-29.2	737.6	7.9	0.7	12.1	597.9	0.4	8.0	205.2	0.5	3.2	52.1	0.2	1.8
Total Q Outflow from Landfill	m3/year								1,627.7								
Leakage Through Engineered Cover (3 mm/year)**	m3/year								1,668.0								
Average Chloride Conc.					38,100					38,100			38,100			38,100	
Chloride Mass	kg/year	0.0	0.0	0.0	28,103.2	302.7	27.8	461.8	22,780.8	17.1	303.9	7,816.8	17.2	122.5	1,984.4	8.2	69.1

(**) Note: Downward seepage through Engineered Cover over the Pre-1986 Landfill and, Cells 16, 17 and 18 (3 mm/year x 556,000 sq m) =1,668 m3/year. The total discharge from the same area is estimated as 1,627.7 m3/year (3.6% lower). This is a round-off error.



Q_{LCS} = 1.0E-7 x [(198.12 mASL - 198.0 mASL) / 3.5 m] x (5.0 m x 1,106.0 m)
= 597.9 m^3/year

Q_D = 2.60E-10 x [(198.12 mASL - 198.0 mASL) / 29.9 m] x (12.3 m x 1,106.0 m)
= 0.4 m^3/year

Q_B = 2.60E-10 x [(198.12 mASL - 198.0 mASL) / 23.7 m] x (192,139.0 m)
= 8.0 m^3/year

$$Q_{(DISCHARGE)} = K \times (\Delta H / L) \times A$$

Where:

K = hydraulic conductivity of medium through which groundwater moves;
ΔH = head differential along groundwater flow path [leachate level (H) - water level at receptor (h)];
L = length of groundwater flow path;
A = cross sectional area through which movement occurs.

LEGEND

198.12 mASL	PROJECTED WATER LEVEL
Q_{LCS}	SHALLOW LATERAL MIGRATION TO THE PERIMETER LCS
Q_D	DEEPER MIGRATION TO INTERFACE AQUIFER
Q_B	VERTICAL MIGRATION TO THE INTERFACE AQUIFER

WATER BALANCE ALTERNATIVE METHOD 1, CELLS 16 AND 17 BUILD OVER

CLEAN HARBORS CANADA, INC.
LAMBTON FACILITY LANDFILL

Drawn by: TFB	Fig: **A.2-3**
Project No.:	1401210
Date Revised:	June 20, 2014

RWDI

Appendix B

Net Effects Analysis for the Existing Landfill (i.e., baseline condition) and the two Alternative Methods incorporating the use of Purge wells for Mitigation

Appendix B.2 – Alternative Method 1

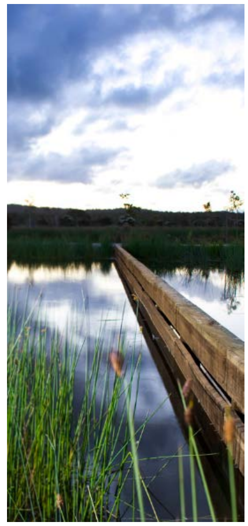
Table B.2-1. Water Balance Use of Purge Wells Lambton Facility Landfill (Alternative Method 1)

		Alternative Method 1 (Vertical Landfill)															
		Constructed over Pre-1986 Landfill							Constructed over Cells 16 & 17			Constructed over Cell 18			Cell 18 (Sub-cell 15)		
		see Figure 5(Cross Section B-B', Figure 1)			see Figure 4 (Cross Section A-A', Figure 1)				see Figure 6 (Cross Section C-C', Figure 1)			see Figure 7 (Cross Section D-D', Figure 1)			see Figure 9 (Cross Section F-F', Figure 1)		
	Units	Shallow Lateral Flow (QLCS and QS)															
Dimensions Considered in Analysis		Constructed over Pre-1986 Landfill**	QS via Active Flow Zone below LCS	QS via Less Active Portion of Active Aquitard	QLCS via Active Flow Zone	QLCS via Less Active Portion of Active Aquitard	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)	Shallow Discharge through Clay Cap and Key (QLCS)	Deep Lateral Discharge across Excavation Sidewall (QD)	Vertical Discharge from Landfill (QB)
Ground Surface Elevation at Toe of Landfill	mASL	202.0	202.0	202.0	200.0	200.0	200.0	200.0	201.0	201.0	201.0	201.0	201.0	201.0	201.5	201.5	201.5
Landfill Properties																	
Height of Landfill on Closure (per Conceptual Design Report)	mASL	203 to 213	203 to 213	203 to 213	203 to 213	203 to 213	203 to 213	203 to 213	203 to 210	203 to 210	203 to 210	203 to 212	203 to 212	203 to 212	202 to 205		
Average Depth of Cell Excavation	m							18.3		18.3	18.3	18.3	18.3	18.3	4.0	18.3	18.3
Landfill Area (A) at Surface	m2							267,345.0			192,139.0			90,484.0			40,950.0
Exterior Perimeter Length of Landfill at Surface	m	388.0	388.0	388.0	1,137.0	1,137.0	1,525.0		1,106.0	1,106.0		1,265.0	1,265.0		506.0	506.0	
Active Aquitard																	
Total Active Aquitard Thickness	m	6.0	6.0	6.0	6.0	6.0	6.0		6.0	6.0		6.0	6.0		6.0	6.0	
Upper Portion of Active Aquitard (Active Flow Zone) Thickness	m	3.0	3.0	3.0	3.0	3.0									3.0		
Hydraulic Conductivity - Active Flow Zone	m/s	2.00E-07	2.00E-07		2.00E-07										2.00E-07		
Lower Portion of Active Aquitard (Less Active Zone) Thickness	m			3.0		3.0									3.0		
Hydraulic Conductivity - Lower Portion of Active Aquitard	m/s			4.00E-09		4.00E-09									4.00E-09		
Effective Hydraulic Conductivity Active Aquitard (where applicable)	m/s														1.02E-07		
Average Elevation of Interface Aquifer Below Landfill	mASL						160	160.0		159.0	159.0		155.0	155.0		161.0	161.0
Clay Aquitard																	
Clay Aquitard Thickness	m							40.0			42.0			46.0			40.5
Clay Aquitard Thickness Adjacent to Waste Cell	m						13.3			12.3			12.3			12.3	
Hydraulic Conductivity - Clay Aquitard	m/s						2.60E-10	2.60E-10		2.60E-10	2.60E-10		2.60E-10	2.60E-10		2.60E-10	2.60E-10
Engineered Features																	
Engineered Cover Thickness	m	2.0	2.0	2.0	2.0	2.0			2.0			2.0	2.0	2.0	2.0		
Width of Clay Plug beyond LCS	m	5.0	5.0	5.0	5.0	5.0			5.0			5.0			5.0		
Depth of Clay Plug	m	1.0	1.0	1.0	4.0	4.0			4.0			4.0			4.0		
Anchor Trench Depth	m	3.0	3.0	3.0	3.0	3.0			3.0			3.0			3.0		
Clay Cover Thickness	m	2.0	2.0	2.0	2.0	2.0			6.1			5.1			5.1		
Clay Key Perimeter Width	m								4.0			5.0			5.0		
Hydraulic Conductivity of Weathered Clay Cover/Clay Key	m/s								1.00E-07			1.00E-07			1.02E-07		
Depth of LCS	m	1.0	1.0	1.0	5.0	5.0			5.0			5.0			5.0		
Invert Elevation of LCS	mASL	201.0	201.0	201.0	195.0	195.0			196.0			196.0			196.5		
Parameters Applied in the Discharge Calculation																	
Water Level Elevation (H, h), expressed in terms of elevation (mASL)																	
Assigned Leachate Level (H)	mASL																
Leachate Level in Waste Cell Estimated by Water Balance (H)	mASL	198.085	198.085	198.085	198.085	198.085	198.085	198.085	198.085	198.085	198.085	198.09	198.09	198.09	198.09	198.09	198.09
Initial Leachate Level in Underlying Waste (H')	mASL														201.0	201.0	201.0
LCS Operating Level (h)	mASL	201.0			198.0	198.0	198.0	198.0	198.0			198.0			198.0		
Ditch Invert (h)	mASL		201.0	201.0	198.0	198.0						199.5					
Simulated Drawdown in Interface Aquifer with purge wells pumping (h)	mASL						197.0	197.0		195.2	195.2		193.2	193.2		194.0	194.0
Distance (L) Along Flow Path to Receptor																	
Separation Distance Top of Waste to LCS	m	2.0	2.0	2.0	2.0	2.0	2.0		2.0	2.0		2.0	2.0				
Average Separation Distance Waste to LCS via Active Flow Zone	m	2.5			3.5				3.5			3.5			22.5		
Average Separation Distance Waste to LCS via Less Active Portion of Active Aquitard	m					6.5			4.5			4.5					
Separation Distance between LCS and Ditch	m		8.0	8.0	8.0	8.0	8.0		8.0	8.0		5.0	5.0				
Clay Cap / Key Width	m								3.5			4.0					
Separation Distance Top of Waste to Ditch	m		10.0	10.0			10.0		10.0	10.0		10.0	10.0				
Average Separation Distance Waste to Ditch via Less Active Portion of Active Aquitard	m		11.5												5.0		
Average Separation Distance Waste to Ditch via Active Aquitard	m			14.5								14.0			20.0		
Average Separation Distance Waste to Ditch via Clay Aquitard	m																
Separation Distance Waste to Interface Aquifer	m						27.4	21.7		29.9	23.7		33.9	27.7		28.4	22.2
Cross Sectional Area (A) for Groundwater Movement																	
Active Flow Zone (lateral flow)	m2	388.0			3,411.0												
Balance of Active Aquitard (lateral flow)	m2		776.0			3,411.0			5,530.0			7,590.0			3,036.0		
Clay Aquitard (lateral flow)	m2			1,164.0			20,282.5			13,603.8						6,223.8	
Landfill Area (vertical flow)	m2							267,345.0			192,139.0		15,559.5	90,484.0			40,950.0
Hydraulic Conductivity Applied along Flow Path	m/s	2.00E-07	2.00E-07	4.00E-09	2.00E-07	4.00E-09	2.60E-10	2.60E-10	1.00E-07	2.60E-10	2.60E-10	1.00E-07	2.60E-10	2.60E-10	1.0E-07	2.60E-10	2.60E-10
Q = K (ΔH/L) A	m3/year	-2,853.43	-1,056.83	-29.52	522.48	5.63	6.60	109.60	423.53	10.78	191.78	145.32	18.41	130.84	36.9	7.4	61.8
Total Q Outflow from Landfill		1,671.0															
Leakage Through Engineered Cover (3 mm/year)**	m3/year	1,668.0															
Average Chloride Conc.		38,100							38,100			38,100			38,100		
Chloride Mass	kg/year	0.0	0.0	0.0	19,906.5	214.4	251.4	4,175.9	16,136.4	410.7	7,306.6	5,536.9	701.5	4,985.0	1,405.6	280.2	2,354.0

(**) Note: Downward seepage through Engineered Cover over the Pre-1986 Landfill and, Cells 16, 17 and 18 (3 mm/year x 556,000 sq m) =1,668 m3/year. The total discharge from the same area is estimated as 1,671 m3/year (0.002% higher). This is a round-off error.

Appendix B

Leachate Mounding Information



Landfill Design Amendments

Supporting Documentation to Variance to ECA A031806

Clean Harbors Canada, Inc.

GHD | 455 Phillip Street Waterloo Ontario N2L 3X2 Canada

044985 | 30 | 02 | Report No 33 | March 9, 2018

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Appendix Index

1. Introduction

Clean Harbors Canada, Inc. (Clean Harbors) is applying for a variance to ECA A031806 which relates to the design and operations of the landfill at the Clean Harbors Lambton Facility that is located at 4090 Telfer Road, St. Clair Township, Ontario (Site). Figure 1.1 provides the Site location with UTM coordinates for the four Site corners.

The request relates to amendments to various design items that are presented in the approved Design and Operations Report – Lambton Landfill Expansion, prepared by Tetra-Tech for Clean Harbors and dated October 8, 2015 (D&O Report). The specific amendments relate to the following items:

a) Hydraulic control layer (Section 6.2.2.1 of D&O Report)

b) Hydraulic connection trenches (Section 6.2.2.2 of D&O Report)

c) Interim Cover (Section 3.4.2 of D&O Report)

d) Cell Development and Cell Sequencing (Section 3.4 of D&O Report)

e) Waste Treatment and Disposal (Section 2.7.2 of D&O Report)

  - Waste Class 331 – Waste Compressed Gases
  - Waste Class 4 – Flammable Solids

f) Hydro-Vac Waste Unloading/Processing Facility

g) Pharmaceutical Waste Disposal

2. Hydraulic Control Layer

The current design of the hydraulic control layer is described in Section 6.2.2.1 and visually presented on Drawing 3 of the Design and Operations (D&O) Report. In the pre-1986 Area, the hydraulic control layer is installed along the base of the new cells to facilitate leachate flow towards the perimeter collection trench. The hydraulic control layer is connected to neighbouring cells to create a continuous permeable layer along the cell base. The current design includes a granular drainage blanket comprised of 0.3 m thick layer of 50 mm clear stone underlain by woven geotextile and overlain with non-woven geotextile. Appendix C.6 of the D&O Report contains the leachate head calculations completed by TetraTech.

The hydraulic control layer installed at the cell base provides the following aspects:

- A continuous preferential pathway to facilitate leachate flow towards the perimeter leachate collection system (LCS), where it can be pumped and managed
- A method to minimize leachate mounding and breakout within the landfill cell

The amendment to the hydraulic control layer is as follows:

1. Decrease the thickness of the clear stone from 0.3 m to 0.15 m.

The hydraulic control layer provides the primary leachate transfer mechanism from the waste to the perimeter LCS during active waste disposal, interim cover period, and the final cover period. The intent of the hydraulic control layer is to minimize the amount of leachate head that occurs on the landfill base and within the waste by providing a free draining condition for the waste during the various operating conditions. During the assessment of the leachate head by GHD, it was noted that the McEnroe equation used in Appendix C.6 assumed free drainage boundary conditions and had an error. Upon investigation, GHD determined the McEnroe equation for free drainage boundary condition in the original paper had an error (Equations 27, 28, and 29). As such, GHD used the general case McEnroe equation (Equations 20, 21, and 22), and confirmed that general case equation produced a smooth curve for the model under various parameters and conditions.

Predicting the maximum leachate head on the landfill base and within the stone drainage layer was completed by using the two equations, the modified USEPA equation and McEnroe equation. The USEPA equation is used in the HELP model and is an analytical solution that provides a reasonable estimate of the maximum leachate head. The McEnroe equation provides a more detailed analytical solution and a more realistic leachate head profile and thus provides a better estimate of the maximum leachate head. Appendix A contains the calculations for the leachate head for both equations, for various scenarios. The appendix also contains a copy of referenced papers. The references for the two equations are as follows:

- USEPA Equation: U.S. Environmental Protection Agency, Washington D.C., 1989. "Requirements for Hazardous Waste Landfill Design, Construction, and Closure". Seminar Publication, EPA-625-4-89-022
- McEnroe Equation: McEnroe. B.M., 1993. "Maximum Saturated Depth over Landfill Liner". Journal of Environmental Engineering, 119:262-270

The modified USEPA and McEnroe methods both assume that infiltration rates reach a steady state condition with regard to leachate generation. Although this is true for the final cover case, this is not the case for the active landfill and the interim cover case, since for both the leachate generation is related to precipitation events and less a steady and uniform generation of leachate over time.

For comparison purposes and to establish a base condition, the McEnroe equation was used to calculate the maximum leachate head for the current 0.3 m thick drainage stone layer, no drainage stone, and a 0.15 m drainage stone layer. The input parameters related to slope, hydraulic conductivity of the stone and waste, the drainage length, etc. are consistent with those used in Tetra-Tech's HELP modelling, provided in Appendix C of the D&O Report. Table 2.1 provides a summary of the maximum leachate head for the three infiltration scenarios, and the drainage layers.

Table 2.1 Maximum Predicted Leachate Head – 2% slope – 300 m length

Scenario	Maximum Leachate Head (m)		
	No Drainage Layer	0.3-thick Drainage Stone Layer	0.15-thick Drainage Stone Layer
Active Fill Period	7.14	1.3	1.3
Interim Cover Installed	2.55	0.37	0.37
Final Cover Installed	0.51	0.06	0.06

Based on the calculations presented in Table 2.1, the use of a drainage stone reduces the maximum leachate head on the base of an order of magnitude for the final cover scenario. The active and interim scenarios indicate that the maximum leachate head will occur in the waste until the final cover is installed.

To assess the landfill operations, the various slopes and a shorter distance were used to assess the predicted maximum leachate head for the Pre-1986 area. A drainage length of 225 m was used for the drainage length, since this is approximately half of the width of the landfill. The equation is not sensitive to the thickness of the drainage stone. For assessment purposes, both thicknesses of drainage stone are presented to indicate if the maximum leachate head occurs within the stone drainage layer. Table 2.2 provides a summary of the three scenarios for three different slopes.

Table 2.2 Assessment of Clear Stone Thickness

Slope	0.5% Slope		1% Slope		2% Slope	
Clear Stone Thickness	0.15 m Stone	0.30 m Stone	0.15 m Stone	0.30 m Stone	0.15 m Stone	0.30 m Stone
Active Fill Period						
Leachate Head (m)	1.82		1.42		0.97	
Maximum Leachate Head within Stone Layer?	×	×	×	×	×	×
Interim Cover Installed						
Leachate Head (m)	0.68		0.46		0.28	
Maximum Leachate Head within Stone Layer?	×	×	×	×	×	✓
Final Cover Installed						
Leachate Head (m)	0.15		0.08		0.04	
Maximum Leachate Head within Stone Layer?	✓	✓	✓	✓	✓	✓

Based on the above, the maximum leachate head will occur within the drainage stone once the final cover has been installed. For the active and interim cover periods, the maximum leachate head will occur in a portion of the waste above the drainage stone.

The drainage stone layer ensures that leachate will be transferred to the perimeter LCS. The modelling indicates that this will occur and that the thickness of stone layer will not impact the amount the leachate transferred, but the amount of leachate that may be mounded in the waste. Extending the maximum leachate height to the groundwater model is not required, since the groundwater model does not assume the 1 m clay base for the vertical landfill expansion is present. Therefore, the existence of the 1 m clay liner and the drainage layer at the landfill provide enhanced leachate collection over above the hydrogeologic model that was used for approval of the landfill.

Based on this, it is recommended that the drainage stone layer in the remainder of the pre-1986 area be reduced from 0.3 m to 0.15 m. The size of the drainage stone, the filter fabrics, and the installation methods should remain the same. Figure 2.1 provides the current approved sections for the hydraulic control layer and the hydraulic connection trench from Drawing G-07 and the proposed amendment section after the reduction in the hydraulic control layer thickness and the removal of the pressure relief boreholes and trench that are discussed in the next section.

3. Hydraulic Connection Trench Amendment

The current design for the hydraulic connection trench is described in Section 6.2.2.2 and visually presented on Drawing 12 and Drawing 14 of the D&O Report. The hydraulic connection trenches/pressure relief boreholes connect the underlying cells (existing landfill) to the hydraulic control layer to control leachate head pressure within the existing landfill cells. Hydraulic control trenches are oriented perpendicular to the perimeter collection trench to convey leachate flow towards the perimeter. The design of the hydraulic control trenches consists of a 0.5 m trench excavated into the existing cap and backfilled with gravel. Pressure relief boreholes are drilled at regular intervals into the underlying waste and filled with drainage stone.

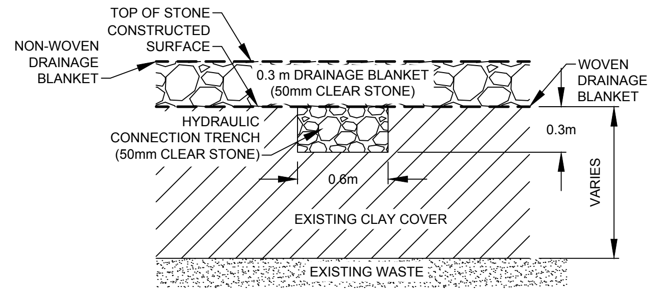
The amendment involves removing the hydraulic connection between the hydraulic control layer and the underlying waste cells through the removal of the hydraulic connection trenches and pressure relief boreholes in the Pre-1986 disposal area (Cells 19-2, 19-3, and 19-4). The design for the vertical expansion portion of the landfill placed above existing landfill Cells 16, 17 and 18 should be reviewed and assessed prior to detailed design of the cells and amendments made prior to landfilling above these cells. The hydraulic connection trenches were intended to facilitate leachate flow towards the perimeter LCS. The following rationale supports the removal of the hydraulic connection trenches.

- The design spacing of the hydraulic connection trenches was based on leachate mounding equations (provided in Appendix C of the D&O Report). As noted in Section 3, the equation used had an error and is considered invalid. Section 2 contains revised calculations for the leachate drainage system that are considered reasonable.
- Based on the grading of the hydraulic control layer, leachate will flow parallel to the hydraulic connection trenches within the hydraulic control layer. The hydraulic connection trenches will not transmit flow towards the perimeter LCS as base grades of the new cells are not sloped towards the trenches. Typically, the leachate trenches, pipes, or perforated material would be placed at low points to collect leachate.

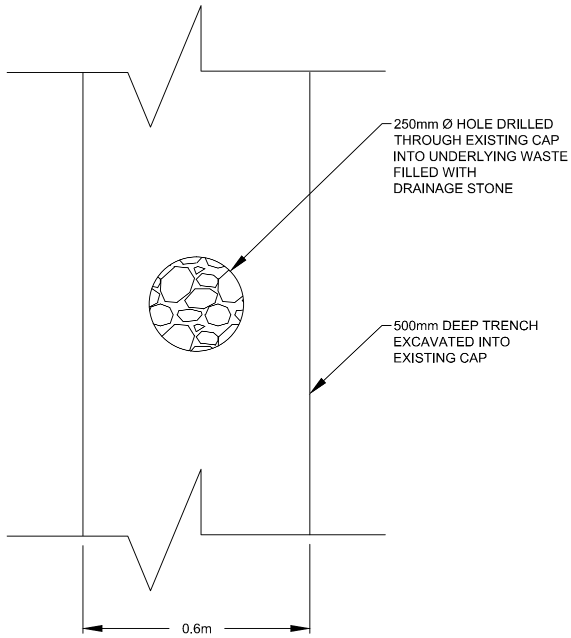
Additionally, the hydraulic control trenches were intended to provide a method for draining of leachate from the underlying waste cells through the pressure relief boreholes. The following rationale supports the removal of the pressure relief boreholes:

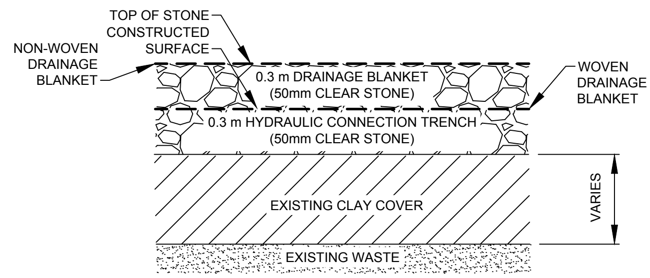
- The design spacing of the pressure relief boreholes was based on the consolidation of underlying waste due to loading and waste placement overtop. The consolidation was estimated to decrease the porosity of the waste by 10 percent (or 1.3 m), which is a conservative approach based on the amount of waste placed at the Site in the noted cells. The settlement was also assumed to occur within 1 year, rather than gradually over time.
- The decrease in porosity due to loading was assumed to cause the leachate to flow under confined pressure. The pressure relief boreholes were designed to accommodate this flow,

APPROVED - DRAWING G-07



SECTION 5 HYDRAULIC CONNECTION TRENCH
NTS G-07

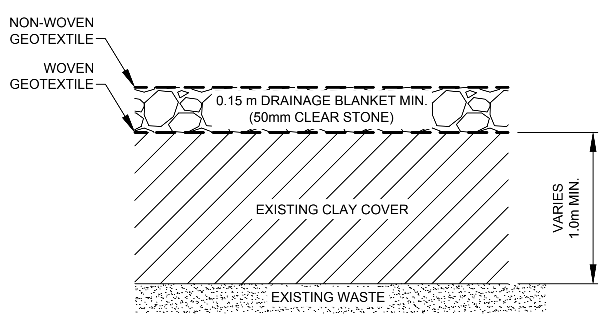




PLAN 4 DRAINAGE STONE CONNECTION DETAIL (TYP)
NTS G-05

DETAIL 6 HYDRAULIC CONNECTION TRENCH
NTS G-07

PROPOSED



SECTION - DRAINAGE STONE
NTS -

CLEAN HARBORS CANADA, INC.
LAMBTON COUNTY, ONTARIO
LANDFILL DESIGN AMENDMENTS

HYDRAULIC CONTROL LAYER AMENDMENTS

44985-30
Feb 14, 2018

FIGURE 2.1

CAD File: P:\drawings\44000s\44985\44985-REPORTS\44985-30(033)\44985-30(033)GN\44985-30(033)GN-WA001.dwg

Appendix A
Maximum Leachate Head Calculations

Appendix A

Maximum Leachate Head Calculations
Landfill Design Amendments Reports
Clean Harbors Lambton Facility

Drainage Length = 225m

Maximum Head (m)

Slope	0.50%		1%		2%	
Method	McEnroe	USEPA	McEnroe	USEPA	McEnroe	USEPA
Active Fill Period						
No Drainage Layer	7.05	6.87	6.38	6.15	5.35	5.24
0.15 m Drainage Stone	1.82	1.76	1.42	1.48	0.97	1.32
0.3 m Drainage Stone	1.82	1.76	1.42	1.48	0.97	1.32
Interim Cover Installed						
No Drainage Layer	3.07	2.96	2.56	2.52	1.91	2.15
0.15 m Drainage Stone	0.68	0.71	0.46	0.64	0.28	0.61
0.3 m Drainage Stone	0.68	0.71	0.46	0.64	0.28	0.61
Final Cover Installed						
No Drainage Layer	0.88	0.89	0.62	0.78	0.39	0.74
0.15 m Drainage Stone	0.15	0.24	0.08	0.23	0.04	0.23
0.3 m Drainage Stone	0.15	0.24	0.08	0.23	0.04	0.23

Appendix A

Maximum Leachate Head Calculations
Landfill Design Amendments Reports
Clean Harbors Lambton Facility

		Active Fill Period					
		No Drainage Layer			Drainage Stone		
		0.5% Slope	1% Slope	2% Slope	0.5% Slope	1% Slope	2% Slope
Parameters	**Unit**						
conductivity (k)	m/s	1.00E-05	1.00E-05	1.00E-05	1.00E-04	1.00E-04	1.00E-04
	m/year	3.16E+02	3.16E+02	3.16E+02	3.16E+03	3.16E+03	3.16E+03
Drainage length	m	225	225	225	225	225	225
Slope (S)	(dimensionless)	0.50%	1.00%	2.00%	0.50%	1.00%	2.00%
Angle (alpha)	rad	0.004999958	0.009999667	0.019997334	0.004999958	0.009999667	0.019997334
L	m	300	300	300	300	300	300
yL	in	0	0	0	0	0	0
	m	0	0	0	0	0	0
YL	(dimensionless)	0	0	0	0	0	0
Parameter							
q	m/yr	0.384	0.384	0.384	0.384	0.384	0.384
q	m/s	1.21682E-08	1.21682E-08	1.21682E-08	1.21682E-08	1.21682E-08	1.21682E-08
R	(dimensionless)	48.67	12.17	3.04	4.87	1.22	0.30
A	(dimensionless)						
B	(dimensionless)	13.91748824	6.904909136	3.342617688	4.297632823	1.966666473	0.466164457
General case solution (Ymax)							
ymax	m	**7.047307991**	**6.383610152**	**5.352585149**	**1.816220762**	**1.418472086**	**0.973846984**
USEPA solution							
ymax	m	**6.873580481**	**6.153630537**	**5.2435703**	**1.756765084**	**1.484893079**	**1.3237325**

Appendix A

Maximum Leachate Head Calculations
Landfill Design Amendments Reports
Clean Harbors Lambton Facility

		Interim Cover Installed					
		No Drainage Layer			**Drainage Stone**		
		0.5% Slope	1% Slope	2% Slope	0.5% Slope	1% Slope	2% Slope
Parameters	**Unit**						
conductivity (k)	m/s	1.00E-05	1.00E-05	1.00E-05	1.00E-04	1.00E-04	1.00E-04
	m/year	3.16E+02	3.16E+02	3.16E+02	3.16E+03	3.16E+03	3.16E+03
Drainage length	m	225	225	225	225	225	225
Slope (S)	(dimensionless)	0.50%	1.00%	2.00%	0.50%	1.00%	2.00%
Angle (alpha)	rad	0.004999958	0.009999667	0.019997334	0.004999958	0.009999667	0.019997334
L	m	300	300	300	300	300	300
yL	in	0	0	0	0	0	0
	m	0	0	0	0	0	0
YL	(dimensionless)	0	0	0	0	0	0
Parameter							
q	m/yr	0.09	0.09	0.09	0.09	0.09	0.09
q	m/s	2.85193E-09	2.85193E-09	2.85193E-09	2.85193E-09	2.85193E-09	2.85193E-09
R	(dimensionless)	11.41	2.85	0.71	1.14	0.29	0.07
A	(dimensionless)						0.845395252
B	(dimensionless)	6.680717568	3.226275311	1.361274651	1.887643696	0.375346824	
General case solution (Ymax)							
ymax	m	**3.071164386**	**2.563217243**	**1.909115874**	**0.676776539**	**0.461643428**	**0.277292031**
USEPA solution							
ymax	m	**2.959601418**	**2.51741398**	**2.154768007**	**0.713770327**	**0.638481713**	**0.611339715**

Appendix A

Maximum Leachate Head Calculations
Landfill Design Amendments Reports
Clean Harbors Lambton Facility

		Final Cover Installed					
		No Drainage Layer			0.15 m Drainage Stone		
		0.5% Slope	1% Slope	2% Slope	0.5% Slope	1% Slope	2% Slope
Parameters	**Unit**						
conductivity (k)	m/s	1.00E-05	1.00E-05	1.00E-05	1.00E-04	1.00E-04	1.00E-04
	m/year	3.16E+02	3.16E+02	3.16E+02	3.16E+03	3.16E+03	3.16E+03
Drainage length	m	225	225	225	225	225	225
Slope (S)	(dimensionless)	0.50%	1.00%	2.00%	0.50%	1.00%	2.00%
Angle (alpha)	rad	0.004999958	0.009999667	0.019997334	0.004999958	0.009999667	0.019997334
L	m	300	300	300	300	300	300
yL	in	0	0	0	0	0	0
	m	0	0	0	0	0	0
YL	(dimensionless)	0	0	0	0	0	0
Parameter							
q	m/yr	0.013	0.013	0.013	0.013	0.013	0.013
q	m/s	4.11945E-10	4.11945E-10	4.11945E-10	4.11945E-10	4.11945E-10	4.11945E-10
R	(dimensionless)	1.65	0.41	0.10	0.16	0.04	0.01
A	(dimensionless)			0.766739904	0.583841841	0.913895763	0.97917772
B	(dimensionless)	2.364590249	0.804950523				
General case solution (Ymax)							
ymax	m	**0.880217394**	**0.622238157**	**0.385714823**	**0.145381615**	**0.083700318**	**0.044635469**
USEPA solution							
ymax	m	**0.894458854**	**0.784246663**	**0.740002293**	**0.23704279**	**0.230658616**	**0.228996279**

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