FUNCTIONAL SERVICING REPORT

DE ZEN INDUSTRIAL LANDS
6678604 ONTARIO INC. & 1105239 ONTARIO INC.

CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

FILE No. 224-M62

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1.0 INTRODUCTION

Skira & Associates Ltd. has been retained to prepare a Functional Servicing Report in support of Draft Plan approval for the proposed development, for the De Zen Industrial Lands – in the City of Mississauga. The lands are described as Part of Lots 11 & 12, Concession 1, West of Hurontario Street.

The total area of the site is approx. 17.57 Ha. The site is bounded by Derrydale Golf Course to the south, an Ontario Hydro One corridor and Highway 407 to the north, existing industrial development and Fletcher's Creek to the west and Derrycrest Drive to the east.
Refer to Figure 1 *for the* ***Site Location Plan.***

The current proposal is to develop the subject land for industrial purposes and will include office buildings and industrial buildings. The Draft Plan area is divided into two (2) blocks and will be developed in two (2) phases. Vehicular access to the site will be through a proposed cul-de-sac to be constructed as per City of Mississauga standards. The purpose of this report is to provide functional servicing design information in support of a Draft Plan of Subdivision application and will demonstrate how the subject lands can be developed in accordance with the City of Mississauga and Region of Peel standards and specifications.

Refer to *the revised* ***Draft Plan*** *prepared by Design Plan Services Inc. in October 2025.*

2.0 BACKGROUND INFORMATION

2.1 Previous Studies, Reports & Planning Documents

The development concepts contained in the report are and extension of, and in accordance with, the information contained in the following reports and engineering drawings:

- Stormwater Management Report entitled *"Fletcher's Creek Business Park"* by Cosburn Patterson Mather Limited (CMP) – October 1999
- Stormwater Management Facility Flow Control Performance Monitoring Report by Sernas – April 2003
- Environmental Impact Study by GEI Consultants – September 2025
- Fluvial Geomorphological Assessment by Parish Geomorphics – September 2025
- Geotechnical Report for Proposed Commercial Development by Soil Engineers Ltd. – September 2025
- Groundwater & Infiltration Study by Soil Engineers Ltd. – September 2025
- Slope Stability Report by Soil Engineers Ltd. – September 2025
- Preliminary Environmental Noise Analysis by Jade Acoustics – October 2014
- De Zen Vicksburgh/Hurontario Traffic Impact Study by GHD – December 2015
- Credit Valley Conservation Authority Stormwater Management Criteria Document – July 2022

2.2 Development Concept

The Draft Plan of Subdivision for the subject lands was prepared by Design Plan Services Inc. in March 2015 and updated October 2025 to reflect new setback limits. The updated Draft Plan forms the basis for the proposed servicing, grading and stormwater management concepts.
Refer to Figure 2 – Proposed Draft Plan of Subdivision.

The subject lands will be accessed by a cul-de-sac extension of the western terminus of the 30-meter right-of-way of Vicksburgh Drive. The cul-de-sac will conform to the limits and grading of the existing dead-end terminus of Vicksburgh Drive. Servicing connections to the subject lands will be provided through connections to existing services within the Vicksburgh Drive right-of-way.

The subject lands are to be developed in two (2) phases. Phase 1 contains the lands west of Derrycrest Drive and east of the Fletcher's Creek Tributary and drainage feature. Phase 2 consists of the lands west of Fletcher's Creek Tributary and drainage feature. These two Phases represent **construction** phases only. The two phases will comprise a single Site Plan application pending Draft Plan approval.

A Scoped Environmental Impact Study (EIS) has been completed by GEI Consultants (March 2014, updated September 2025) in support of the Draft Plan application to document and evaluate existing environmental site conditions. The study involved consultation with the City of Mississauga, Credit Valley Conservation (CVC), and the Region of Peel in an effort to define the proposed limits of development for the site. The limit of development was then used to prepare the proposed Draft Plan. The findings of the EIS have been used to limit potential impacts/disturbance as it relates to both construction and post-construction activities on site.

As part of the proposed development, the existing wetland north of the remnant pond will be retained as it provides significant wildlife habitat, as discussed in the EIS. Industrial road crossing will be provided in order to maintain access connection between both phases of the project. With a proposed 2.4m box culvert, drainage, crossing, detail design will be further defined to met CVC criteria for waterway crossing.

A large portion of the site will remain undeveloped (5.95 Ha). This portion consists of the natural features and setbacks associated with Fletcher's Creek Development Limits and Fletcher's Creek tributary as outlined by GEI Consultants in the September 2025. This area will be deeded gratuitously to the City as greenbelt for conservation purposes as shall be appropriately rezoned per City of Mississauga.

This report provides a general servicing strategy for the subject lands. Final details related to site servicing will be finalised at the Subdivision Design and Site Plan approval stages.

3.0 EXISTING SITE CONDITIONS

3.1 Land Use

The site is located in the Credit Valley Conservation watershed within Fletcher's Creek Subwatershed.

The subject lands have been previously used for agriculture purposes. External drainage areas north of the site support an existing wetland which drains into an agricultural pond in the centre of the site. The agricultural pond drains to a watercourse that discharges into Fletcher's Creek.

Fletcher's Creek abuts the western and southern limits of the Site. The stream corridor is outside the development limits. ***Refer to Figure 3 – Existing Conditions Plan***.

3.2 Soil Conditions

Based on the findings of the Soil Investigation (Soil Engineer Ltd. – September 2025), the site is covered by topsoil underlain by a silty clay till deposit. Completed test pits show topsoil thickness ranging from 0.25m to 0.35m which contains roots and humus.

The predominant soil type within the existing site surface is hard silty clay till. Ground water was detected 4.6m below the ground surface or at an approx. elevation of 196.00m. For the purposes of hydrologic analysis, the soil was classified as hydrologic soil Group BC.

The Geotechnical Report and construction recommendations can be found in ***Appendix A***.

3.3 Topography

The majority of subject lands consist of a gently sloping plateau subdivided by a valley containing a tributary to Fletcher's Creek. The subject lands descend gently from the eastern boundary at Derrycrest Drive and the Hydro One Corridor. The western and southern edges of the site slope steeply towards Fletcher's Creek.

The Existing Site Conditions and Pre-Development Drainage Area Plan (respectively ***Figure 3*** and ***Figure 6***) include information from a Topographical Survey completed in January 2015.

3.4 Groundwater Conditions

Appendix A Pavement Infiltration & Groundwater Report for the De Zen Industrial Lands completed by Soil Engineers Ltd. on September 2025. Two boreholes were drilled to depths of 5.8m and 6.1m. Beneath a layer of topsoil, the site is generally underlain by a stratum of silty clay till. Groundwater was detected at depths of 3.8m and 4.6m relative to the existing ground elevations.

The results of their analysis, in the Groundwater Infiltration Testing by Soils Engineers Ltd – September 2025, shows that the testing parameters for the groundwater samples within the site are below the laboratory detection limits or within Table 2 criteria in a potable groundwater condition under the EPA.

3.5 Slope Stability Assessment

Appendix A includes a Slope Stability Report from September 2025. This document details the results of a slope stability assessment performed by Soil Engineers Ltd. These results show that the slope at cross-sections B-B to E-E has a factor of safety (FOS) ranging from 1.79 to 2.40, which satisfies the OMNR guideline requirements for infrastructure and public land uses.

Cross-Section A-A, just downstream of the junction between Fletcher's Creek and the Fletcher's Creek Tributary, has a FOS of 1.45, which fails to meet the OMNR requirements. As per the Soil Engineers Ltd. Report, the slope should be re-graded with a gradient of 1V:2H as is recommended for use in sound native clay till with CVC permission as this area is located within the dedicated Fletcher's Creek area/. The remodelled slope yields a FOS of 1.55 which meets the OMNR requirements. The long-term slope stability limit has been considered in establishing the development setbacks for the subject lands.

4.0 GRADING

The preliminary site grading for the subject lands has been designed to minimize disturbance to existing boundaries, match the existing perimeter and generally follow the existing topography.

Refer to Figure 4 – Composite General Site Grading Plan.

Phase 1 of the subject lands will be graded such that major system drainage from the area drains towards Derrycrest Drive. Major drainage system will be piped in this scenario.

The cul-de-sac detailed in ***Figure 5*** will be graded such that it aligns precisely with the existing limits of Vicksburgh Drive. Minor system drainage from the cul-de-sac will be collected through catchbasins within the cul-de-sac, while major system drainage will be conveyed overland towards the 100-year capture point on the subject lands.

Phase 2 of the subject lands will be graded such that the major and minor flow drainage is directed towards the Fletcher's Creek valley. Major drainage system will be controlled on site within the loading areas of industrial buildings and piped to the outlet.

All internal roads will have asphalt pavement complete with concrete curbs and gutters designed and constructed in accordance with the latest O.P.S. and/or City standards and requirements.

Detailed grading for the subject lands will be provided in future submissions for Site Plan approval.

5.0 STORM DRAINAGE SYSTEM

5.1 Existing Storm Drainage

The site is located in a large subwatershed that originates from south of HWY 407 and the Hydro One corridor and discharges into Fletcher's Creek south of Derry Road (Stormwater Management Report, CPM, October 1999).

The CPM report identifies the subject lands as being contained entirely within Subwatershed No. 101, which has a total area of 52.9 Ha. This area encompasses all the land east of the Fletcher's Creek Greenbelt, west of Hurontario Street, North of Derry Road and south of the 407. The report considers the entirety of Subwatershed 101 as a contributing drainage area to the Fletcher's Creek SWM Pond located south of Derry Road. However, under existing conditions the subject lands drain overland to Fletcher's Creek.

The subject lands receive 12.16 Ha external drainage from the Hydro One corridor, Hydro One Lands, and a small portion of Hurontario Street right-of-way boulevard through a drainage feature that connects the external lands to the remnant farm pond as shown in ***Figure 6***. The external drainage is then conveyed from the remnant farm pond to Fletcher's Creek through a tributary herein labelled Tributary 1.

As shown in ***Figure 6***, pre-development Catchments 1 and 2 drain to Tributary 1 via the drainage feature and remnant farm pond and future bridge. Catchments 3 and 4 drain directly to Fletchers Creek below the bridge structure. External Catchments Ext.5, 6 & 7 currently drain overland to Derrycrest Drive.

Application has been filed by De Zen Construction for development of areas Ext. 5 & 6. These areas are included in the calculation to drainage feature as a temporary conservative measure until development proceeds.

Drainage information for contributing external areas north of the site and drainage areas internal to the subject lands is included in **Table 5.1** below.

Area F1 and F2 are not included in the analysis as their area contributes drainage to Fletcher's Creek basin directly outside of drainage feature.

Table 5.1 – Pre-Development Drainage Areas

Area ID	Area (ha)	$C_{initial}$	$C(100yr)_{adjusted}$	%IMP	CN	CN (AMCIII)	IA (mm)	Slope (%)	TP (hr)
Ext. 1	2.55	0.90	1.00	1.00	77	89	2	0.47	0.38
Ext. 2	4.18	0.25	0.31	0.07	65	81	8	1.27	0.33
Ext. 3	0.88	0.50	0.63	0.43	71	85	5	1.38	0.22
Ext. 4	0.87	0.25	0.31	0.07	78	89	8	1.06	0.29
temp Ext.5	1.80	0.25	0.31	0.07	75	88	8	1.22	0.28
Ext.6	1.78	0.25	0.31	0.07	75	88	8	1.16	0.14
A1	2.60	0.25	0.31	0.16	77	89	8	0.84	0.32
A2	1.94	0.25	0.31	0.70	77	89	8	2.41	0.10
A3	2.37	0.25	0.31	0.07	77	89	8	0.58	0.30
A4	2.04	0.25	0.31	0.07	77	89	8	2.63	0.15
A5	1.81	0.25	0.31	0.07	77	89	8	4.60	0.10

***Area F1 (3.27) and F2 (3.55) contribute directly to Fletcher's Creek drainage.**

The time-to-peak for all catchments was determined using the Upland Method. According to a OTTHYMO analysis of the above catchments, the governing peak flow from the external lands into the site under existing condition is 1.46m³/s and is generated by the 100-yr AES storm. These results are included in ***Appendix B*** and summarized in **Table 5.1.1** below.

Table 5.1.1 – Pre-Development External Drainage Peak Flows by Storm Type

Storm Event	Peak Flow (m³/s)
100-yr 12-hr SCS	1.17
100-yr 4-hr Chicago	1.46
Regional (Hurricane Hazel)	1.01

5.2 Proposed Land Use

The subject lands will be developed for industrial purposes. As shown on ***Figure 2***, the subject lands will be developed as a single industrial commercial block common element condominium of 11.68 Ha. Due to the single site access location on Vicksburgh Drive and the tributary that Fletcher's Creek tributary running through the centre of the site, the block area will be developed in two phases as shown on ***Figures 7 & 8***. These two phases will have distinct servicing strategies for storm and sanitary servicing. Phase 1 (3.81 Ha) and Phase 2 (7.75 Ha) will consist of industrial buildings and office uses. The number and footprint of the buildings within both phases will be determined at the Site Plan approval stage and layout shown on report figures is conceptual. Vehicular access to the site will be provided from the intersection of Vicksburgh Drive and Derrycrest Drive.

A cul-de-sac will be constructed at the western end of Vicksburgh Drive according to City of Mississauga standards and for assumption by the City of Mississauga Standard No. 2211.250. Dimensions of this cul-de-sac are included in ***Figures 4 & 5***.

Private access driveway from Vicksburgh ROW to west side of the creek will be designed as common element condominium road including culvert crossing.

5.3 Proposed Storm Drainage

5.3.1 External Lands

The external drainage to the site under existing conditions consists of 12.16 Ha of drainage from the north, including Hydro One Lands, vacant agricultural lands, and a small portion of Hurontario Street. Under existing conditions, a drainage feature conveys the external drainage areas to the remnant farm pond in the centre of the subject lands. As certified in the 2011 Sernas SWM memo included in ***Appendix E***, the area north of the Hydro One land does not convey drainage to the subject lands.

Development of the subject lands will not require the redirection or any drainage from **external lands**. Under proposed conditions, this drainage will continue to be conveyed through the existing wetland and the existing remnant farm pond, and existing watercourse lands dividing Phase 1 & Phase 2.

Figure 7 shows that under proposed conditions, external catchments 5 and 6 will be redirected to the Vicksburgh Drive storm sewer. This reflects the ultimate build-out conditions of the external area according to the Subdivision 678604 Ontario Inc. T-11001 report submitted by Lethbridge and Lawson in 2011. These ultimate build-out conditions are corresponding with ultimate condition runoff coefficient per the Mississauga Stormwater standards of the Mississauga Design Requirements.

The ultimate condition of external catchments 5 and 6 will reduce the external flows to the wetland under ultimate conditions. For the purposes of a more conservative design, the larger pre-development external flows were used to determine the sizing of the culvert. The pre-development peak flow used to design the constructed culvert are included in ***Appendix B*** and summarized in ***Table 5.1***.

According to this analysis, the 100-year 12-hour SCS storm produced the highest peak flow at 2.00m³/s. This design flow was used to determine the size of the proposed culvert. An 18000 x 1200 concrete box pipe will be constructed. Details related to the design and sizing of the culvert are included in ***Appendix B***.

5.3.2 Development Lands Conveyance

The minor storm system is a series of storm sewers generally sized to convey the 10-year return period storm in the City of Mississauga. A preliminary storm design sheet showing proposed contributions to infrastructure has been completed for the development and are included in ***Appendix B***. ***Figure 7*** (Post-Development Storm Drainage Plan) provides an overview of the Phase 1 and Phase 2 drainage areas that will contribute to minor system conveyance. Details of the minor system collection and conveyance for Phase 1 and Phase 2 will be determined at the Site Plan Approval Stage.

Phase 1

The minor system servicing for Phase 1 will capture and convey the 100-year flow eastward towards the existing storm sewer on Vicksburgh Drive and ultimately to a stormwater management pond. The 100-yr peak flow from Phase 1 will be captured and conveyed in order to ensure all storm flows up to and including the 100-yr storm receive quality control in the stormwater pond south of Derry Road (City of Mississauga Pond 4402B).

The design sheet is included in ***Appendix B*** describing the existing storm servicing on Vicksburgh Drive and Derrycrest Drive. The design sheet reflects the servicing with the proposed Phase 1 contribution. The Lethbridge and Lawson design sheet reflects the original drainage contributions for which the Derrycrest sewer was designed. As shown, the proposed Phase 1 peak flow of 1.356m³/s is well below the peak flow rate for which the sewer was designed (3.014m³/s from the north section connection). The discrepancies in these design sheets reflect updates in the proposed servicing that have occurred since the original design sheet was produced.

Phase 2

The minor storm system for Phase 2 will capture and convey the 10-yr peak flow towards the existing Fletcher's Creek to the west. The preliminary outlet for Phase 2 has been established during site visit with CVC and GSI Consultants in 2018. *The proposed design sheet for Phase 2 outlet has been provided in **Appendix B**.*

Major system is piped and will utilise large loading and parking areas surfaces to control runoff from development areas and convey flows towards outlet location.

Note: Internal storm sewers within the proposed site plan for Phases 1 & 2 might need to be designed to a 100-yr storm intensity of overland flow route cannot be achieved through grading. Details will be established during site plan approval process.

6.0 STORMWATER MANAGEMENT

Stormwater management practices are planning and technical measures which will be implemented to manage the quality and quantity of urban runoff. The proposed stormwater practices for this development will be designed in accordance with the recommendations and criteria outlined in both the Fletcher's Creek Master Drainage Plan, the City of Mississauga Development Requirements Manual (Transportation and Works Department, 2024) and the Stormwater Management Planning and Design Manual (Ministry of Environment and Climate Change, 2003).

6.1 Stormwater Management Criteria

6.1.1 Quantity Control

The City of Mississauga Development Requirements Manual (effective December 2018) state that no quantity control is required for developments in this portion of the Fletcher's Creek Subwatershed. In addition to the City criteria, Ministry of Transportation requires stormwater quantity control to 2-yr pre-development flow for 100-yr storm event.

6.1.2 Quality Control

Since Fletcher's Creek is the downstream receiver for the sites stormwater runoff and is defined as a Type 1 habitat in the Fish habitat Protection Guidelines for Developing Area (MNR, 1994), "Enhanced" or Level 1 water quality protection (80% TSS removal) is required. Therefore, quality control measures to achieve this level of protection will be applied to SWM servicing in accordance with the SWM Manual (MOECC, 2003).

6.1.3 Erosion Control

The Fletcher's Creek established criteria for erosion control for Pond 4402B as the detention of runoff from a 25 mm storm for 24 hours which is consistent with MOECC guidelines. However, within the MOECC SWM Manual guidelines, the minimum criterion for Active Storage Detention is 12 hours if the active storage detention is in conflict with the minimum orifice size. Under the proposed drainage strategy, Phase 1 only will be conveyed to Pond 4402B.

The CVC also states that erosion control for receiving watercourses that are not sensitive, i.e. Tributary 1 of Fletcher's Creek, can be achieved through detention/retention of the first 5mm of rainfall. This level of erosion control will be provided for Phase 2.

6.1.4 MTO Quantity Control

The area adjacent to the Highway 407 ETR corridor is subject to Ministry of Transportation for quantity control. On site stormwater management will be implemented in order to ensure that the 100-yr post-development flows will not exceed allowable 24hr pre-development flow.

6.2 Stormwater Management Design

The Stormwater management of the subject lands will consist of two distinct stormwater management strategies for Phase 1 and Phase 2 as described below.

Phase 1

6.2.1 Quality Control & Erosion Control

Stormwater drainage from Phase 1 up to and including the 100-year peak flow will be conveyed to and treated within Pond 4402B south of Derry Road. The stormwater management facility south of Derry Road has been designed to provide Level 1 Enhanced (80% TSS Removal) water quality protection, 24 hour detention of the 25 mm design storm and post to pre water quantity control for the entirety of Subwatershed 101 which includes Phase 1 and Phase 2 of the subject lands. The facility is operating according to the design (Stormwater Management Facility Flow Control Performance Monitoring Report, Sernas, 2003). ***See Appendix E.***

The proposed development will convey less drainage to the Fletcher's Creek SWM facility. The pond designed including the Phase 2 lands under our new development proposal, will be discharged to Fletcher's Creek after receiving quality and erosion control instead of being directed towards the SWM pond.

The total flow from Phase 1 is far less under the proposed drainage plan than was originally accounted for in the CPM report due to the redirection of Phase 2 into the Fletcher's Creek Tributary.

See ***Appendix B*** for the flows accounted for in the Lethbridge and Lawson report ($3.014 m^3/s$) and the flows that results from the proposed development of Phase 1 at C = 0.90 – City of Mississauga 2018 standard runoff coefficient for industrial areas – ($1.623 m^3/s$). The storm sewer design sheets in ***Appendix B*** show that the existing storm sewers on Vicksburgh Drive and Derrycrest Drive have sufficient capacity to accommodate the 100-yr peak flow from Phase 1.

Phase 2

6.2.2 Quality Control

Since the proposed development will not require an on-site wet pond for quantity control, other methods of achieving the required water quality control (i.e. Enhanced Level 1) have been considered. Although stand-alone end-of-pipe treatment solutions (i.e. Oil/Grit Separator Units) are manufactured to treat areas up to 5 Ha, it is understood that such strategies are not typically supported by the CVC as the only end-of-pipe solution. With this in mind, the proposed strategy to achieve the required water quality requirement will be through the use of a "treatment train" approach which relies on the cumulative benefits gained from using a combination of SWM practices (i.e. lot level, conveyance and end-of-pipe).

The proposed treatment train approach may involve a combination of the following and will be finalized at the detailed design stage:

1. Lot-Level/LID BMP Treatment (Retention of first 5mm of rainfall)
   a) Increased topsoil depth
   b) Dedicated clean-water conveyance from rooftops
   c) Catchbasin Treatment – Goss Traps or CB shields

2. End-of-Pipe Level Treatment:
   a) OGS Unit
   b) Erosion control storage tank with filtration (Cultec-type system)
   c) Bio-Retention Swale

Section 6.3 provides more detail on the LIDs that were considered.

6.2.3 Erosion Control

A number of erosion control strategies have been considered for Phase 2. These include the following:

- Controlled rooftop storage
- Erosion control storage tanks

Predevelopment discharge was established for erosion control 25mm storm using 7.75 Ha and T_C = 15 min.

$$Q = 7.75 \times 40.06 \times 0.25 / 360$$
$$= 0.215 m^3/s$$

Erosion control discharge using 215 L/s will exceed required 24-hour release. In order to provide required 24 hour release rate, the discharge rate needs to be reduced to achieve the required 24 hr. release combined with erosion storage. A release rate of Q = 0.018m³/s was selected.

This flow rate was used as the discharge rate to calculate volume required for the erosion control storage volume for 25mm storm event. The storage required to maintain this flow rate under post-development conditions was determined to be 1532.65m³. A rational method model was used to determine sizing of the erosion volume.

YEAR STORM
25mm EROSION
CITY
MISSISSAUGA

C = 0.900
A (ha) = 7.75000
Allow. Discharge Qa (m3/s) = 0.018000
Safety Factor Sf = 0.00%

Max. Required Detention (m3) = ***1532.65***

RAINFALL DURATION *Tc (min)*	RAINFALL INTENSITY *I (mm/hr)*	TOTAL UNCONTROLLED RUNOFF *Q=CIA/360 (m3/sec)*	INFLOW VOLUME Vi (m3)	OUTFLOW VOLUME Vo (m3)	REQUIRED DETENTION VOLUME (m3) *D=(Vi-Vo)*Sf*
5	78.13	1.5137	454.10	5.42	448.68
10	56.33	1.0915	654.88	10.80	644.08
15	44.77	0.8675	780.71	16.18	764.53
20	37.50	0.7266	871.88	21.56	850.33
25	32.46	0.6289	943.39	26.93	916.45
30	28.74	0.5568	1002.30	32.31	969.99
35	25.87	0.5012	1052.50	37.70	1014.81
40	23.58	0.4568	1096.32	43.08	1053.24
45	21.70	0.4205	1135.26	48.46	1086.80
50	20.14	0.3901	1170.35	53.84	1116.51
55	18.80	0.3643	1202.34	59.22	1143.12
60	17.66	0.3422	1231.76	64.60	1167.16
65	16.66	0.3228	1259.03	69.99	1189.04
70	15.78	0.3058	1284.46	75.37	1209.09
75	15.01	0.2907	1308.30	80.75	1227.54
80	14.31	0.2772	1330.76	86.14	1244.62
85	13.68	0.2651	1352.00	91.52	1260.48
90	13.12	0.2541	1372.16	96.91	1275.25
95	12.60	0.2441	1391.36	102.29	1289.07
100	12.13	0.2349	1409.69	107.68	1302.01
105	11.69	0.2265	1427.23	113.06	1314.17
110	11.29	0.2188	1444.06	118.44	1325.62
115	10.92	0.2116	1460.24	123.83	1336.41
120	10.58	0.2050	1475.83	129.21	1346.61
125	10.26	0.1988	1490.86	134.60	1356.26
130	9.96	0.1930	1505.38	139.99	1365.39
135	9.68	0.1876	1519.43	145.37	1374.06
140	9.42	0.1825	1533.04	150.76	1382.29
145	9.17	0.1777	1546.25	156.14	1390.11
150	8.94	0.1732	1559.07	161.53	1397.54
155	8.72	0.1690	1571.53	166.91	1404.62
160	8.51	0.1650	1583.66	172.30	1411.36
165	8.32	0.1612	1595.47	177.69	1417.79
170	8.13	0.1575	1606.99	183.07	1423.91
175	7.95	0.1541	1618.22	188.46	1429.76
180	7.79	0.1509	1629.18	193.84	1435.34
185	7.63	0.1477	1639.89	199.23	1440.66
190	7.47	0.1448	1650.36	204.62	1445.75
195	7.33	0.1419	1660.61	210.00	1450.60
200	7.19	0.1392	1670.63	215.39	1455.24
205	7.05	0.1366	1680.45	220.78	1459.68
210	6.92	0.1341	1690.07	226.16	1463.91
215	6.80	0.1317	1699.51	231.55	1467.96

220	6.68	0.1295	1708.76	236.94	1471.82
225	6.57	0.1272	1717.84	242.33	1475.51
230	6.46	0.1251	1726.75	247.71	1479.04
235	6.35	0.1231	1735.51	253.10	1482.41
240	6.25	0.1211	1744.11	258.49	1485.62
245	6.15	0.1192	1752.56	263.87	1488.69
250	6.06	0.1174	1760.87	269.26	1491.61
255	5.97	0.1156	1769.05	274.65	1494.40
260	5.88	0.1139	1777.10	280.04	1497.06
265	5.79	0.1123	1785.02	285.42	1499.59
270	5.71	0.1107	1792.81	290.81	1502.00
275	5.63	0.1091	1800.49	296.20	1504.29
280	5.55	0.1076	1808.06	301.59	1506.47
285	5.48	0.1062	1815.51	306.97	1508.54
290	5.41	0.1048	1822.86	312.36	1510.50
295	5.34	0.1034	1830.11	317.75	1512.36
300	5.27	0.1021	1837.26	323.14	1514.12
305	5.20	0.1008	1844.31	328.52	1515.78
310	5.14	0.0995	1851.26	333.91	1517.35
315	5.07	0.0983	1858.13	339.30	1518.83
320	5.01	0.0971	1864.90	344.69	1520.21
325	4.95	0.0960	1871.59	350.08	1521.52
330	4.90	0.0949	1878.20	355.46	1522.73
335	4.84	0.0938	1884.72	360.85	1523.87
340	4.78	0.0927	1891.17	366.24	1524.93
345	4.73	0.0917	1897.54	371.63	1525.91
350	4.68	0.0907	1903.84	377.02	1526.82
355	4.63	0.0897	1910.06	382.41	1527.65
360	4.58	0.0887	1916.21	387.79	1528.42
365	4.53	0.0878	1922.29	393.18	1529.11
370	4.48	0.0869	1928.31	398.57	1529.74
375	4.44	0.0860	1934.26	403.96	1530.30
380	4.39	0.0851	1940.14	409.35	1530.80
385	4.35	0.0842	1945.97	414.74	1531.23
390	4.30	0.0834	1951.73	420.12	1531.61
395	4.26	0.0826	1957.43	425.51	1531.92
400	4.22	0.0818	1963.08	430.90	1532.18
405	4.18	0.0810	1968.67	436.29	1532.38
410	4.14	0.0803	1974.20	441.68	1532.52
415	4.10	0.0795	1979.68	447.07	1532.61
420	4.07	0.0788	1985.10	452.46	**1532.65**
425	4.03	0.0781	1990.47	457.84	1532.63
430	3.99	0.0774	1995.80	463.23	1532.56
435	3.96	0.0767	2001.07	468.62	1532.45
440	3.92	0.0760	2006.29	474.01	1532.28

This volume will be provided within the cultec chambers located and underground storage pipes in front of each building. A combined volume of 1,554.5m³ will be provided in two (2) locations as shown on ***Figure 5***.

Time of Retention Release:

T = 1532m³ / 0.018m³/s
= 85,111s (23.64 hrs.)

Erosion Controls

An erosion controls release rate will be achieved using orifice restrictor plate within Splinter MH 2 located at the southwest corner before discharge.

The orifice size is 77mm dia. using high water elevation of 2.08m above the centroid of the orifice opening. *See **Appendix C** for orifice calculations.*

6.3 Potential Additional LID Measures

Based on in-situ testing provided by Soil Engineers, for the areas of infiltration at cultec system, the infiltration rate is 7mm/hr, which is less then the permitted MECP criteria for infiltration.

In addition to the cultic bottomless trench, LID BMPs, such as permeable paving and potential bioswale rain gardens, represent possible applications for the De Zen Industrial Lands. These LIDs will be further explored for feasibility at the Site Plan Approval stage. Note that most of the runoff that will be infiltrated by LID BMPs will not be retained on site for recharge due to the imperviousness of the underlying soil; flows will instead be temporarily attenuated in the soils but will ultimately be either captured in the storm sewer system or discharged as interflow into Fletcher's Creek or the Fletcher's Creek tributary. Infiltration measures will promote short term attenuation and filtration within the native material. The potential LID measures will be designed to provide a minimum of 5mm runoff retention **where feasible**, enhance quality/erosion control, and to promote evapotranspiration. LID BMPs will also be used to meet the "Enhanced" or Level 1 water quality protection (80% TSS removal) using a treatment train approach. LID BMPs will be considered for pre-treatment, conveyance, and end-of-pipe stormwater management solutions.

LID BMPS for Consideration:

- Increased topsoil depth in landscaped areas along the perimeter of the site
- Rain gardens/bioretention within landscape areas
- Localised permeable paving within the parking areas of industrial buildings

6.4 External Feature Water Balance

A wetland water balance analysis was performed based on the preliminary Draft Plan in order to ensure that the proposed wetland receives a quantity of runoff equal to or greater than the existing wetland. The EIS report completed by GSI Consultants (January 2025), confirmed that the wetland is not a PSW and the remnant pond is not a significant feature.

The redirection of runoff from the Phase 1 area to Vicksburgh Drive and Phase 2 to Fletcher's Creek, runoff to the wetland is expected to decrease under post-development conditions. Introduction of uncontrolled landscape areas along the limits of the north and south section of the wetland area will improve post-development conditions. *For detail calculations **refer to Appendix C**.*

6.4.1 Phase 1 – 5mm Water Balance

Existing infiltration is not significant as the site is not within a groundwater recharge area and predominantly consists of soft to hard silty clay till (Soil Engineer Ltd. Report, September 2025). Permeable pavement designs will only be feasible on parking area due to heavy truck loads on the private roads and the potential for groundwater contamination.

Using impervious are as, 3.87 Ha required 5mm runoff to be retained on site is as follows:

V_{5mm} = 38,700 x 0.005
= 193.50m³

Parking areas in front of Buildings D, E & F, G will be constructed as permeable surface. Approx. 2,100m³ of permeable paving is suggested to meet minimum.

$V_{PROVIDED}$ = 2,100 x 0.30 x 0.4
= 252m³

Where, 0.30m represents typical pavement thickness, and
0.40m represents porosity of storage medium.

6.4.2 Phase 2 – 5mm Water Balance

Using impervious areas of this portion of the development approx. 7.75 Ha and coverage typical for industrial buildings. Required 5mm runoff to be retained on site is as follows:

V_{5mm} = 77,500 x 0.005 x 0.90 (impervious surface)
= 348.75m³

Although the cultec system gravel base would provide excellent exfiltration, the in-situ soil testing confirmed infiltration will not be possible due to ground conditions.

Similar to Phase 1, areas of permeable paving will be introduced to provide required volume.

Approx. 3,000m², located at visitor parking area, in front of Buildings A, B & C, will provide required storage volume to meet minimum. Locations of permeable paving will be shown and detailed at Site Plan approval.

$V_{PROVIDED}$ = 3,000 x 0.3 x 0.4
= 360m³

Where, 0.30 represents standard granular pavement depth, and
0.40 porosity of clear stone storage medium.

Based on the hydrogeological information provided by Soil Engineers Ltd. for this area, the hydraulic conductivity for the silty clay layers is approx. $1.0E^{-7}$. *See **Appendix A***.

The expected drawdown time for the infiltration cell was calculated using Equation 4.3 of the MECP Stormwater Management Planning & Design Manual.

Based on Equation 4.3:

(Time to Infiltrate)

$$\Delta t = \frac{1000V}{APn}$$

Where, A = bottom of trench area
V = volume to be infiltrated
n = porosity
P = percolation rate of native soil
Δt = retention time

Using the worst-case scenario, bottom surface of the permeable paving at elevation is approx. 0.4m below proposed surface and 12mm/hr assumed empirical value for silty clay type soil at this elevation, and groundwater surface 196.20 (4.0m below):

$$\Delta t = \frac{1000 \times 360m^3}{3000m^2 \times 0.4 \text{ (porosity)} \times 12mm/hr / 2.5 \text{ (safety factor)}}$$

Δt = **62.5 hrs** – is acceptable time for infiltration by MECP

During the detailed design process, other LID measures (e.g. rain gardens/bio-retention, increased topsoil depth, filter strips, attenuation galleries/infiltration swales, clean water conveyance from rooftops and perforated pipe systems) might be employed to provide quality control to Phase 2 and to mitigate any potential reduction in recharge.

*Please refer to **Appendix D** for **Water Balance Calculations**.*

6.5 Quality Control – Oil/Grit Interceptor

In addition to LID measures presented in **Section 6.4**, the storm sewer runoff conveyed to the storm sewer outlet at Fletcher's Creek through the erosion trench will be directed to the oil/grit interceptor structure to provide initial pre-treatment.

Normally, these facilities operate based on principle of sedimentation of the grit and phase separation for the oil. They are most suitable for institutional/commercial/industrial areas where the level of concentrated pollutants is expected to be higher than residential areas. Being an industrial development, it is considered feasible to provide and oil/grit separator (OGS) on the storm sewer line.

The stormwater runoff from the site will be intercepted and conveyed though the OGS prior to being discharge to outlet to Fletcher's Creek.

The proposed oil/grit separator is Type HD8 Hydrodome manufactured by CIP Hydroworks. The proposed oil/grit interceptor will provide greater efficiency of the TSS removal, estimated at 82%.

The design principles for this type of separator (manhole-type) are as follows: Low flows enter a lower chamber where sedimentation and oil separation can occur. High flows will bypass the low chamber, flowing through the upper chamber directly to the outlet pipe.

6.6 Quantity Control

Area = 7.77 Ha

6.6.1 Allowable Discharge

Allowable discharge from the area of development at pre-development conditions will be established using Phase 2 area.

$Q_{2\text{-yr}}$ = 7.75 x 0.25 x 59.89 / 360
= 0.322m³/s

$Q_{100\text{-yr}}$ = 7.75 x 0.25 x 140.69 / 360
= 0.757m³/s

Where, A = Area
C = 0.25 (pre-development runoff coefficient)
I_2 = 59.89
I_{100} = 140.69

YEAR STORM
100 YEAR
CITY
MISSISSAUGA

C = 0.950
A (ha) = 7.70000
Allow. Discharge Qa (m³/s) = 0.323000
Safety Factor Sf = 0%

Max. Required Detention (m³) = ***2948.90***

RAINFALL DURATION *Tc (min)*	RAINFAL.L INTENSITY *I (mm/hr)*	TOTAL UNCONTROLLED RUNOFF *Q=CIA/360 (m³/sec)*	INFLOW VOLUME Vi (m³)	OUTFLOW VOLUME Vo (m3)	REQUIRED DETENTION VOLUME (m³) *D=(Vi-Vo)*Sf*
5	242.53	4.9282	1478.45	99.29	1379.16
10	176.31	3.5826	2149.54	193.80	1955.74
15	140.69	2.8587	2572.86	288.49	2284.38
20	118.12	2.4002	2880.22	383.30	2496.92
25	102.41	2.0809	3121.39	478.20	2643.19
30	90.77	1.8445	3320.08	573.17	2746.91
35	81.77	1.6616	3489.32	668.20	2821.12
40	74.58	1.5154	3636.96	763.28	2873.68
45	68.68	1.3956	3768.12	858.41	2909.71
50	63.75	1.2954	3886.29	953.58	2932.71
55	59.56	1.2103	3993.93	1048.78	2945.16
60	55.95	1.1369	4092.90	1144.01	**2948.90**
65	52.81	1.0730	4184.58	1239.27	2945.32
70	50.03	1.0167	4270.05	1334.55	2935.50
75	47.58	0.9667	4350.17	1429.86	2920.31

Detention volumes available are as follows:

CB No.	Catchbasin Top Elevation (m)	100-yr Ponding Elevation (m)	100-yr Ponding Depth (m)	100-yr Storage Available (m^3)
Building A Loading	200.70	201.20	0.50	1,409
Building B	200.70	201.20	0.50	755
Building C	200.70	201.20	0.50	765
			Total:	**2,929**

Allowable volumes on loading areas satisfy storage requirements.

6.6.2 Orifice Control

The max. allowable runoff release rate of **0.322m^3/s** will be achieved by the means of an orifice restrictor tube installed over the outlet pipe at Splitter MH STMMH 2 located at the property line southwest corner. The size of the orifice restrictor pipe is **247mm dia.**

Erosion control plate 77mm dia. will provide 25mm runoff controls and pond up to 2.08m depth. A spill zone will be provided within manhole to allow higher intensity flows to cross. *Detail of splitter manhole and orifice locations have been included in **Appendix C**.*

The orifice discharge rate was calculated using FlowMaster computer program developed by Haestad Methods Inc. (USA) and an output report is attached.

7.0 WASTEWATER SERVICING

7.1 Existing Wastewater Infrastructure

The Vicksburgh Drive existing sanitary sewer flows to a 300mm existing sanitary sewer on Derrycrest Drive. Ultimately, sanitary flows are conveyed to the GE Booth (Lakeview) Waste Water Treatment Plant (WWTP) on Lakeshore Road East, Mississauga.

To the west of the subject lands, a 1500mm sanitary trunk sewer flows from north to the south along the path of Fletcher's Creek. The sanitary trunk sewer lies within a 10-metre easement (Part 3, Plan 43R-22904), ***see Appendix E*** *for Region of Peel Drawings*, which crosses through the south-western corner of the subject lands. *Refer to* ***Figure 8*** *for more information related to sanitary servicing.*

7.2 Wastewater Servicing Design Criteria

Wastewater infrastructure will be designed in accordance with the latest Region of Peel standards and specifications as follows:

Wastewater Design Criteria

Average Dry Weather Flow:	302.8 litres per capita per day
Infiltration:	0.26 litres per second per hectare
Population:	70 persons/hectare or equivalent

7.3 Proposed Wastewater Servicing

Internal sanitary sewers will be constructed along the proposed driveways. Individual service connections will be provided for each building within the development, according to the criteria established by the Region of Peel. Two separate servicing strategies are proposed for the areas to the west and east of the proposed wetland. These are referred to as Phase 1 and Phase 2.

Phase 1

Phase 1 sanitary flows will be conveyed by gravity to the existing 300mm sanitary sewer on Vicksburgh Drive. Sanitary flows will then be conveyed eastward to a 300mm sanitary sewer on Derrycrest Drive.

Existing sanitary connection has been already constructed on Derrycrest Drive and will be utilised for connection of Building D. New proposed connection will be provided at terminus of Vicksburgh Drive to service Building C.

Phase 1 Industrial Area – 4.33 x 70p/hectares = 303 population

$$\text{Peak Factor} = 1 + \frac{14}{4 + 0.303^{0.5}}$$

$$= 1 + 3.07$$

$$= 4.07 \simeq \text{(max. 4.0 factor)}$$

Expected Peak Flow Rate = 302.8 x 303 x 4.0
= 366,993.6 L/day = 4.25 L/s

Infiltration Flow = 4.33 x 0.26 L/s/Ha
= 1.12 L/s

Design Flow = 4.25 + 1.12
= 5.37 L/s

Phase 2

A variety of alternatives have been considered for sanitary servicing of the Phase 2 lands. The alternatives were assessed according to the existing infrastructure availability and Region of Peel standards as well as environmental issue related to the wetland preservation identified by Savanta and Credit Valley Conservation.

It had been determined that the optimal servicing option for Phase 2 sanitary is to convey westward by gravity to the existing 1500mm trunk sanitary sewer to the west of the site (Part 3, Plan 43R-22904). This will require the least intensive earthworks operations and will utilize existing infrastructure.

Phase 2 Industrial Area – 7.75 x 70p/hectares = 542 population

Peak Factor $= 1 + \frac{14}{4 + 0.542^{0.5}}$
$= 1 + 2.95$
$= 3.95$

Expected Peak Flow Rate = 302.8 x 542 x 3.95
= 648,264.5 L/day = 7.50 L/s

Infiltration Flow = 7.75 x 0.26 L/s/Ha
= 2.01 L/s

Design Flow = 7.50 + 2.01
= 9.51 L/s

Sanitary Design Sheets are included in ***Appendix D, drawings SS-1 Site Servicing Plan & SAN-1 Sanitary Drainage Plan.***

7.4 Access Easement

A 10m wide access easement across Phase 1 and 2, traversing east to west, will be provided to Region of Peel to access Regional trunk sewer easement structures. Retail location of the easement will be secured through the road surface across the culvert and parking surface and R-Plan will be submitted for Region review at detail design.

8.0 WATER DISTRIBUTION

8.1 Existing Water Supply System

The existing 300mm diameter watermain along Vicksburgh Drive is the intended service connection for the development site. The proposed site is located in Pressure Zone 5 of Lorne Park Water Treatment Plant.

There are two existing fire hydrants in the immediate vicinity of the proposed development; one on Derrycrest Drive south west of the subject lands and one on Vicksburgh Drive east of the subject lands.

Existing 200mm watermain connection on the west side of Derrycrest Drive will be utilised to provide servicing to Building D.

Hydrant flow tests was performed in Oct, 2025 and results shown in the Appendix G

8.2 Proposed Water Demand Criteria

Water servicing for the subject lands will be designed in accordance with the latest Region of Peel standards and specifications to achieve adequate pressure and fire flows.

8.3 Proposed Water Demand

Phase 1 & Phase 2 Industrial as per previously established:

Total Employment Population	= 303 + 544 = 847
Total Expected Peak Flow	= 300 x 847 x 3.0 = 762,300 L/day = 8.82 L/s
Total Expected Max. Daily Flow	= 300 x 847 x 1.40 = 355,740 L/day = 4.11 L/s

8.4 Internal Servicing

The subject lands will be serviced by an existing 300mm diameter watermain on Vicksburgh Drive. Preliminary water servicing design includes an internal watermain layout that follows the alignments of the proposed driveways, with connections to the existing 300mm diameter watermain on Vicksburgh Drive. Individual service connections will be provided to the proposed buildings within the development from the watermain on the fronting driveways.

8.5 Fire Flow

Based on the Fire Underwriter Survey 2020, the fire flow is calculated on the area of largest industrial building floor using the following formula: $F = 220 \sqrt{A} \times C$

Where, C = Coefficient of fire resistance construction = 0.60
A = Area = 13,570 (floor area high one-storey building)
F = Fire Flow in L/m

F = 220 x 0.60 x √13,570
= 14,800 L/min

Calculated value can be reduced by 40% if automatic sprinkler system is provided. Therefore,

F = 14,800 x 0.40
= 8,880 L/min (148 L/s)

Therefore, Max. Daily Flow = 148 + 4.11
= **152.11 L/s**

Fire flow was conducted on the existing watermain and confirms that existing storm can provide sufficient domestic and fire flows. *See Applied Fire Flow Test in **Appendix G**.*

8.6 Utilities

Existing utility services are available on Vicksburgh Drive. Bell/Cable/Hydro and Enbridge will extend their existing services to accommodate the De Zen development.

9.0 EROSION & SEDIMENT CONTROL

The erosion and sediment control plan for the subject lands will be designed at the Site Plan Approval stage. Prior to any land stripping or regrading within the subject lands, an Erosion and Sediment Control Permit will be obtained from the City of Mississauga and Conservation Halton as part of the Site Alteration Process.

The Erosion and Sediment Control Plan will be designed in conformance with the City of Mississauga, Credit Valley Conservation Authority and MOECC guidelines. Erosion and Sediment Controls will be implemented for all construction activities including topsoil stripping, foundation excavation and stockpiling of materials.

The Erosion and Sediment Control strategy will consider the implementation of the following measures:

- Temporary sediment control fence at construction limits and/or downstream of any disturbed areas prior to grading. Double row fencing may be required adjacent to sensitive natural areas;
- Gravel mud mats at construction vehicle access points to minimize off-site tracking of sediments;
- Temporary sedimentation control ponds;
- Conveyance controls including but not limited to cut-off swales;
- Check dams, etc. for erosion / velocity control;
- Temporary stabilization measures (e.g. erosion blankets)
- Sediment traps in catchbasins;
- Routine inspection, monitoring, and repair as necessary of all temporary Erosion and Sediment Control measures during construction; and,
- Removal of temporary controls once the areas they serve are restored and stable.
- Construction runoff will be directed away from proposed LID facilities; after site is vegetated, erosion and sediment control structures will be removed.

The following erosion and sediment control measures will be installed and maintained during construction:

- A temporary sediment control fence will be placed prior to grading.
- Sediment traps will be provided.
- Gravel mud mats will be provided at construction vehicle access points to minimize off-site tracking of sediments.
- All temporary erosion and sediment control measures will be routinely inspected and repaired during construction. Temporary controls will not be removed until the areas they serve are restored and stable.

All reasonable measures will be taken to ensure that sediment loading is minimized both during and following construction.

10.0 SUMMARY & CONCLUSIONS

This Functional Servicing Report provides the framework to address the required infrastructure associated with the proposed Draft Plan. Based on the foregoing analysis and discussions it is concluded that:

The preliminary grading analysis completed is consistent with the pre-development drainage boundaries and is in harmony with adjacent lands.

The planning, preliminary grading, servicing, and stormwater management strategies presented in this Functional Servicing Report demonstrate that it is now appropriate to proceed with the Draft Plan of Subdivision approval for the subject lands.

10.1 Storm Servicing

Phase 1

- Stormwater runoff for Phase 1 up to and including the 100-year peak flow will be conveyed to the existing 1200mm diameter storm sewer located east of the site.
- Stormwater quantity, quality, and erosion control for Phase 1 will be provided by the existing SWM facility located at the south of Derry Road.

Phase 2

- Stormwater runoff for Phase 2 will be conveyed by the major and minor system toward the west Fletcher's Creek outlet.
- Stormwater quality and erosion control for Phase 2 will be provided by a combination of OGS's and LID treatment train BMPs.

10.2 Sanitary Servicing

Phase 1

- The existing 300 mm diameter gravity sewer along Vicksburgh Drive has sufficient capacity to service Phase 1.

Phase 2

- Phase 2 sanitary drainage will be directed westwards to the 1500mm sanitary trunk sewer to the west of the subject lands. The sanitary sewer connecting Phase 2 to the trunk sanitary sewer will be constructed so as to avoid all nearby environmentally sensitive areas.

10.3 Water Servicing

Water supply to Phase 1 and Phase 2 of the subject lands will be provided by the existing 300mm diameter Regional watermain along Vicksburgh Drive.

We respectively submit this report with intention of obtaining approval in principle the recommendation.

Yours truly,

SKIRA & ASSOCIATES LTD.

Michael Jozwik, P. Eng.
MJ:ak

NOTE: ***Limitation of Report***

*This report was prepared by **Skira & Associates Ltd.** for **678604 Ontario Inc. & 1105239 Ontario Inc.** for review and approval by government agencies only.*

*In light of the information available at the time of preparation of this report, any use by a **Third Party** of this report are solely the responsibility of such **Third Party** and **Skira & Associates Ltd.** accepts no responsibility for any damages, if any, suffered by the **Third Party**.*

APPENDIX A

SLOPE STABILITY LETTER
AND
INFILTRATION REPORT
BY: SOIL ENGINEERS LTD.

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL • ENVIRONMENTAL • HYDROGEOLOGICAL • BUILDING SCIENCE
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September 18, 2025 Reference No. 2507-S026
(Revision of Report dated April 13, 2020)

De Zen Realty Company Limited
4890 Tomken Road, Units 1-4
Mississauga, Ontario
L4W 1J8

Attention: Mr. Mark Palmieri

Re: Supplementary Slope Stability Study Letter Report
Proposed Employment Lands
De Zen Industrial - Phase 2
Southwest of Highway 407 and Hurontario Street
City of Mississauga

Dear Sir:

In 2008, a soil investigation consisting of 4 boreholes to depths ranging from 4.9 to 7.9 m was carried out onsite for a slope stability study. Subsequent to the 2008 slope stability report, Reference No. 0803-S002, an addendum was issued in 2012 to provide additional analyses and clarifications to address the Credit Valley Conservation (CVC) comments dated February 28, 2012. The topographic map for the site has since been updated. The previously analyzed cross-sections are therefore revised accordingly.

The slope stability report was revised again in 2016 and 2020 to address comments issued in 2015 by the CVC, and to incorporate the toe erosion allowance (TEA) and meander belt width (MBW) prepared and presented by GEO Morphix Ltd. in their 'Tributary of Fletcher's Creek Erosion Hazard Assessment', draft dated April 15, 2019. The MBW has since been refined in 2025, and in response, we herein present our supplementary slope stability study findings and recommendations, incorporating the updated setbacks.

FINDINGS

Based on the 2008 borehole information, beneath a layer of topsoil, 15 to 30± cm thick, the site is underlain by a layer of generally hard silty clay till and very dense sandy silt till.

All boreholes remained dry upon completion of field work. However, a groundwater level of El. 195.0± m was included in the modeling at the request of CVC and was assumed to taper towards Fletcher's Creek. In the absence of monitoring well data, the use of the transition zone where the colour of the soil changes from brown to grey best represents the potential groundwater regime.

SLOPE STABILITY STUDY

The slope stability study focuses on the eastern bank of Fletcher's Creek, meandering along the western and southern limits of the subject site. The drainage feature downstream to the pond in the centre of the site has been identified as a watercourse by CVC and therefore has been added to the slope study. At the time of the 2008 inspection, the drainage ditch was dry.

Cross-Sections A-A to E-E, were selected to represent the most critical portions of the slope. The locations of the cross-sections are shown on Drawing No. 1. These sections have an overall slope height of 3.0± to 8.0± m, measured from the tableland to the toe of slope, with an overall gradient of 1V:1.9 ± to 3.4± H and a local gradient of 1V:0.9 H. The surface profiles of the cross-sections are interpreted from the contours on the topographic plan prepared by David B. Searles Surveying Ltd.; the subsurface profiles are interpreted from the borehole logs. Cross-Sections A-A to E-E are shown on Drawing Nos. 2 to 8, inclusive.

As noted in the previous report and letter, visual inspection revealed that the slope is generally well-vegetated with dense grass- and weed-covers and sparse trees in the northern region where the slope is gentle. In the southern region where the slope is the steepest, tree growth was more prominent. No signs of seepage or major deep-seated failure were observed; however, minor channelization and surface creeping were noted in the proximity of Cross-Section B-B. In addition, active toe erosion was observed in the absence of a flood plain along the creek bank at Cross-Sections A-A and B-B (Boreholes 1 and 2). No active erosion was noted along the drainage/gulley features.

The slope stability was analyzed using limit-equilibrium criteria of the Bishop Method with the effective soil strength parameters shown in the table below.

Strength Parameters for Slope Stability Analysis

	γ (kN/m^3)	c' (kPa)	ϕ' (degrees)
Silty Clay Till	22.0	5	30
Sandy Silt Till	22.0	0	31

The result from the analysis indicates that the slope at Cross-Sections B-B to E-E has a factor of safety (FOS) ranging from 1.79 and 2.40, which satisfies the OMNR guideline requirements for infrastructure and public land uses (minimum FOS of 1.5). These existing slopes are therefore considered geotechnically stable. The results are presented on Drawing Nos. 4, 6, 7 and 8.

For Cross-Section A-A, the result (Drawing No. 2) shows that the existing slope has a FOS of 1.45, which fails to meet the OMNR requirements. A stable gradient of 1V:2.2H is recommended for use in the sound native clay till. The remodelled slope, yielding a FOS of 1.54 which meets the OMNR requirements, is presented on Drawing No. 3.

In the absence of an adequate flood plain (less than 15 m in width), a TEA of 5 m has been recommended by GEO Morphix Ltd. in their study. This is mainly applicable for Cross-Sections A-A, B-B and E-E and surrounding areas. For Cross-Sections B-B and E-E, a geotechnically stable gradient of 1V:1.9H to 1V:2H is used behind the TEA. The remodelled slopes, with FOS of 1.57 and 2.12, meets the OMNR requirements and is presented on Drawing Nos. 5 and 9.

The Long-Term Stable Slope Line (LTSSL), incorporating the geotechnically stable gradients and TEA, where applicable, is established on the Borehole and Cross-Section Location Plan, Drawing No. 1. For the most part, the LTSSL coincides with the Top of Bank (staked with CVC on November 9, 2001) or the Farm Pond Drainage Area (staked July 6, 2012). The LTSSL is not defined for the unconfined feature east/opposite of Cross-Section E-E (also known as Reach FCT-2); instead, the MBW will govern in this area.

Lastly, a development setback buffer for man-made and environmental degradation of the bank will be required, subject to the discretion and approval of CVC.

In future development, should any alteration be carried out in the slope areas, it should either be restored to its original condition or better than its original condition.

In order to prevent the occurrence of localized surface slides in the future and to enhance the stability of the slope, the following geotechnical constraints should be stipulated:

1. The prevailing vegetative cover must be maintained, since its extraction would deprive the rooting system that is reinforcement against soil erosion by weathering. If for any reason the vegetation cover is stripped, it must be reinstated to its original, or better than its original, protective condition. Restoration with selective native plantings including deep rooting systems which would penetrate the original buried topsoil

shall be carried out to ensure bank stability.

2. Grading of the land adjacent to the slope must be such that concentrated runoff is not allowed to drain onto the slope face. Landscaping features which may cause runoff to pond at the top of the slope must not be permitted.
3. The leafy topsoil cover on the bank face should not be disturbed, since this provides insulation and a screen against frost wedging and rainwash erosion.
4. Where development is carried out near the top of the slope, there are other factors to be considered related to possible human environmental abuse. Soil saturation from maintenance of landscaping features, stripping of topsoil or vegetation, and dumping of loose fill over the bank must not be allowed.

The above recommendations are subject to the approval of the CVC.

We trust this letter satisfies your present requirements; however, should any queries arise, please feel free to contact this office.

Yours truly,
SOIL ENGINEERS LTD.

Hui Wing Yang, P.Eng.
HWY/BL

Bernard Lee, P.Eng.

ENCLOSURES

Borehole and Cross-Section Location Plan	Drawing No. 1
Cross-Section A-A (Existing Condition)	Drawing No. 2
Cross-Section A-A (Stable Condition)	Drawing No. 3
Cross-Section B-B (Existing Condition)	Drawing No. 4
Cross-Section B-B (Stable Condition)	Drawing No. 5
Cross-Section C-C (Existing Condition)	Drawing No. 6
Cross-Section D-D (Existing Condition)	Drawing No. 7
Cross-Section E-E (Existing Condition)	Drawing No. 8
Cross-Section E-E (Existing Condition with Toe Erosion Allowance)	Drawing No. 9

c. Soil Engineers Ltd. (Mississauga) Attn: Mr. Benjamin Lee

SKETCH ILLUSTRATING
TOPOGRAPHIC DETAIL
DEZEN INDUSTRIAL LANDS PHASE 2
CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

SCALE 1: 500

0 5 10 15 20 25 30 40 50 metres

David B. Searles Surveying Ltd.
ONTARIO LAND SURVEYORS

METRIC
DISTANCES AND COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048

BENCHMARK NOTE

ELEVATIONS ARE REFERRED TO THE CITY OF MISSISSAUGA BENCHMARK No. 1079 BEING A BRASS MONUMENT AT THE BASE OF A CONCRETE CONTROL MONUMENT AT THE SOUTHWEST CORNER OF THE INTERSECTION OF DERRYCREST DRIVE AND VICKSBURGH DRIVE, HAVING AN ELEVATION OF 150.110m.

VERTICAL DATUM CANADIAN GEODETIC DATUM, 1928
(GVD 1978 SOUTHERN ONTARIO READJUSTMENT)

LEGEND

FH	DENOTES FIRE HYDRANT
BC	DENOTES BACK OF CURB
BF	DENOTES BOARD FENCE
CF	DENOTES CHAIN FENCE
COUT	DENOTES CURB CUT
CLF	DENOTES CHAIN LINK FENCE
CMP	DENOTES CORR. METAL PIPE
CSP	DENOTES CULVERT (STEEL PIPE)
CSW	DENOTES CONCRETE SIDE WALK
DF	DENOTES DRAINAGE FEATURE
EW	DENOTES EDGE OF WATER
LS	DENOTES LIGHT STANDARD
MH	DENOTES MAINTENANCE HOLE COVER
MH(CMW)	DENOTES MAINTENANCE HOLE COVER (CONTAMP)
MH(STM)	DENOTES MAINTENANCE HOLE COVER (STORM)
PWF	DENOTES POST AND WIRE FENCE
VP	DENOTES VENT PIPE
WP	DENOTES WOOD POLE
WV	DENOTES WATER VALVE
x	DENOTES BOTTOM OF SLOPE
	DENOTES DITCH LINE
	DENOTES TOP OF SLOPE
	DENOTES CONIFEROUS TREE
	DENOTES DECIDUOUS TREE
	DENOTES TREE LINE

BLOCK 70
REGISTERED PLAN 43M-1788
PIN 14080-1757(LT)

PART 2 PLAN 43R-19029 SUBJECT TO EASEMENT AS IN RO650229

BLOCK 172
PIN 14080-1657(LT)
REGISTERED PLAN 43M-1760

BLOCK 150
REGISTERED PLAN 43M-1759
PIN 14080-1473(LT)

BLOCK 171 REGISTERED PLAN 43M-1760 PIN 14080-1656(LT)

PART 3, PLAN 43R-22604

PART 2 SUBJECT TO EASEMENT AS IN RO650840

PART 1 SUBJECT TO EASEMENT IN GROSS AS IN PR1298835

PART 2 SUBJECT TO EASEMENT IN GROSS AS IN PR1298834

PART 1, PLAN 43R-31088
PART 1, PLAN 43R-9576
PART 3 PLAN 43R-31088
PART 4 PLAN 43R-31088 SUBJECT TO EASEMENT IN GROSS AS IN PR1271071
PART 7 PLAN 43R-9576
PART 3 PLAN 43R-9576
PART 4 PLAN 43R-9576 PIN 14080-2040(LT)

LIMIT OF PARAMENT BELT BY GEO MORPHIX

BH1
BH2
BH3
BH4

LONG-TERM STABLE SLOPE LINE

LOT 12
PART 2, PLAN 43R-34093
PIN 14080-2039(LT)

CONCESSION 1 WEST OF HURONTARIO STREET

LOT 11
PART 4 , PLAN 43R-31437
PIN 14080-2046(LT)

5-M TOE EROSION ALLOWANCE BY GEO MORPHIX

MEANDER BELT BY GEO MORPHIX

MEANDER BELT BY GEO MORPHIX (FCT-2)

PART 3, PLAN 43R-31437

PART 2 SUBJECT TO EASEMENT AS IN RO626587

PART 6 PLAN 43R-31437 SUBJECT TO EASEMENT AS IN INST No. RO626587

PART 2 , PLAN 43R-19739
PIN 14080-1174(LT)

PART 1 PLAN 43R-32715

PART 3 PLAN 43R-34093 SUBJECT TO RIGHT OF WAY AS IN RO650864

PART 7 PLAN 43R-31437 SUBJECT TO EASEMENT AS IN TT82416

BLOCK 7 0.30 RESERVE PIN 14080-2054(LT)

BLOCK 8 0.30 RESERVE PIN 14080-2053(LT)

BLOCK 9 0.30 RESERVE PIN 14080-2052(LT)

BLOCK 11 0.30 RESERVE

BLOCK 6 0.30 RESERVE DERRYCREST DRIVE

VICKSBURGH DRIVE (BY REGISTERED PLAN 43M-1874) PIN 14080-2062(LT)

PART 1 PLAN 43R-33517 PIN 14080-2041(LT)

PART 2 PLAN 43R-33517 VICKSBURGH DRIVE BY BY-LAW 0126-2012 AS IN PR2223697 (SCHEDULE 'A') PIN 14080-2041(LT)

PART 3 PLAN 43R-33517 PIN 14080-2041(LT)

DERRYCREST DRIVE (BY REGISTERED PLAN 43M-1874) PIN 14080-2063(LT)

REGIS

PIN 14080-2040(LT)

FLETCHERS CREEK

TOP OF BANK AUG 6, 2012

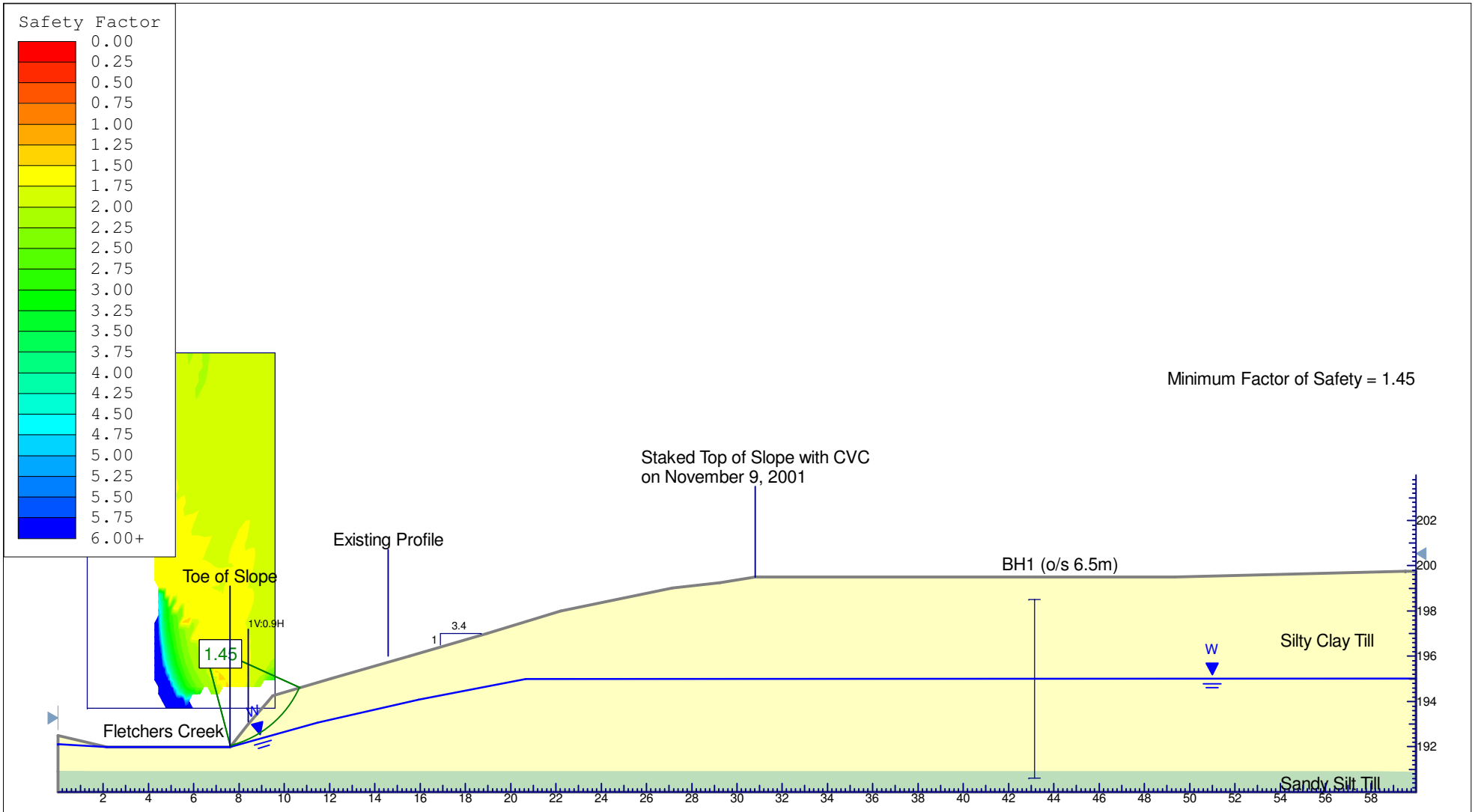
DETERMINATION OF MAN MADE POND STATUS JULY 12, 2012

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BOREHOLE & CROSS-SECTION LOCATION PLAN
DE ZEN INDUSTRIAL PHASE 2

SITE: SOUTHWEST OF HIGHWAY 407 AND HURONTARIO STREET
CITY OF MISSISSAUGA

DESIGNED BY:	**CHECKED BY:**	**DWG NO.:** 1	
SCALE: 1:2000	**REF. NO.:** 2507-S026	**DATE:** SEPTEMBER 2025	**REV** D



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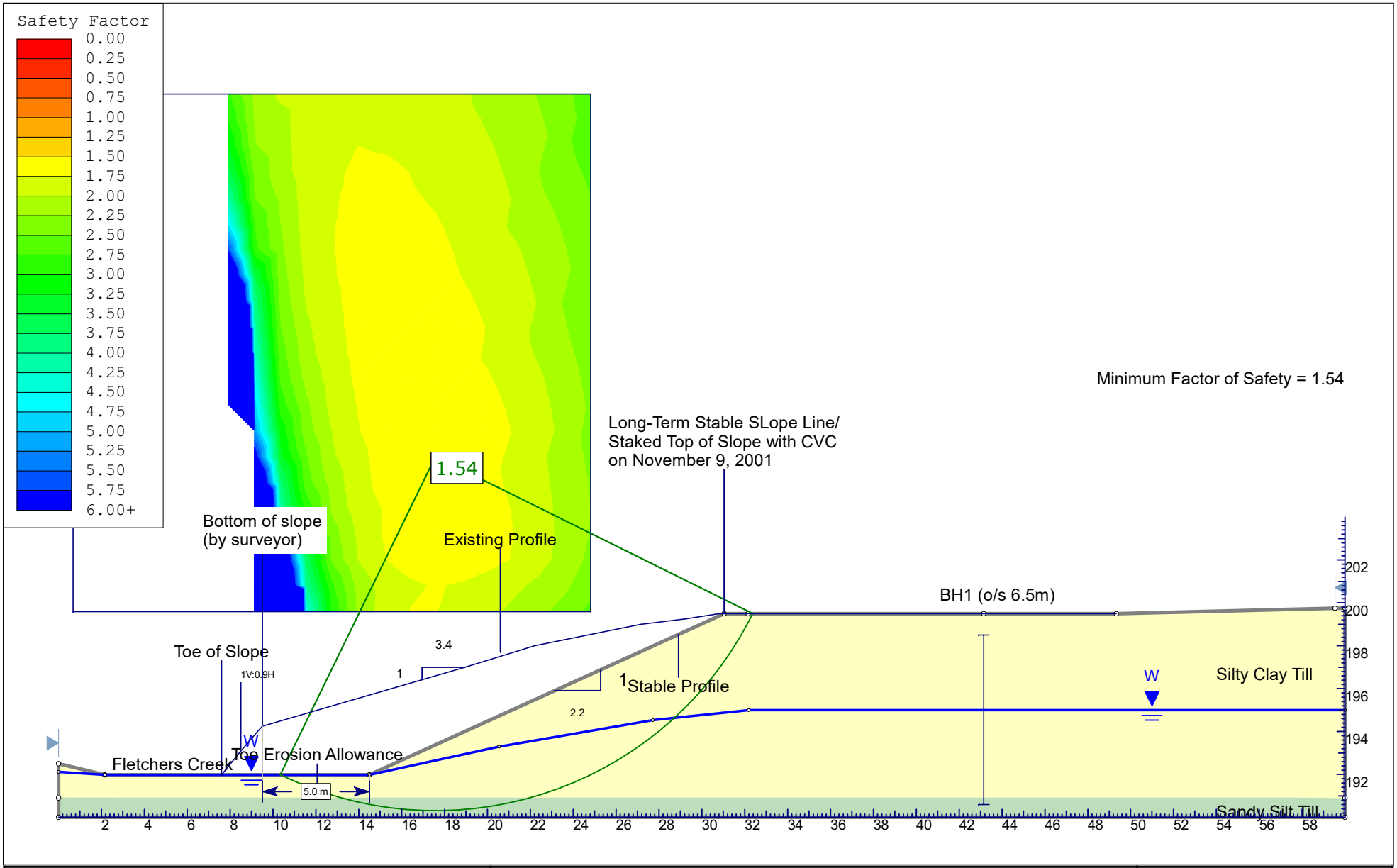
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 · TEL: (416) 754-8515 · FAX: (416) 754-8516

Project Title: Cross-Section A-A			Load Case: Existing Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: BL	Checked By:	Scale: 1:250	Revision: -
Date: January 2016		Reference No.: 0803-S002	Drawing No.: 2



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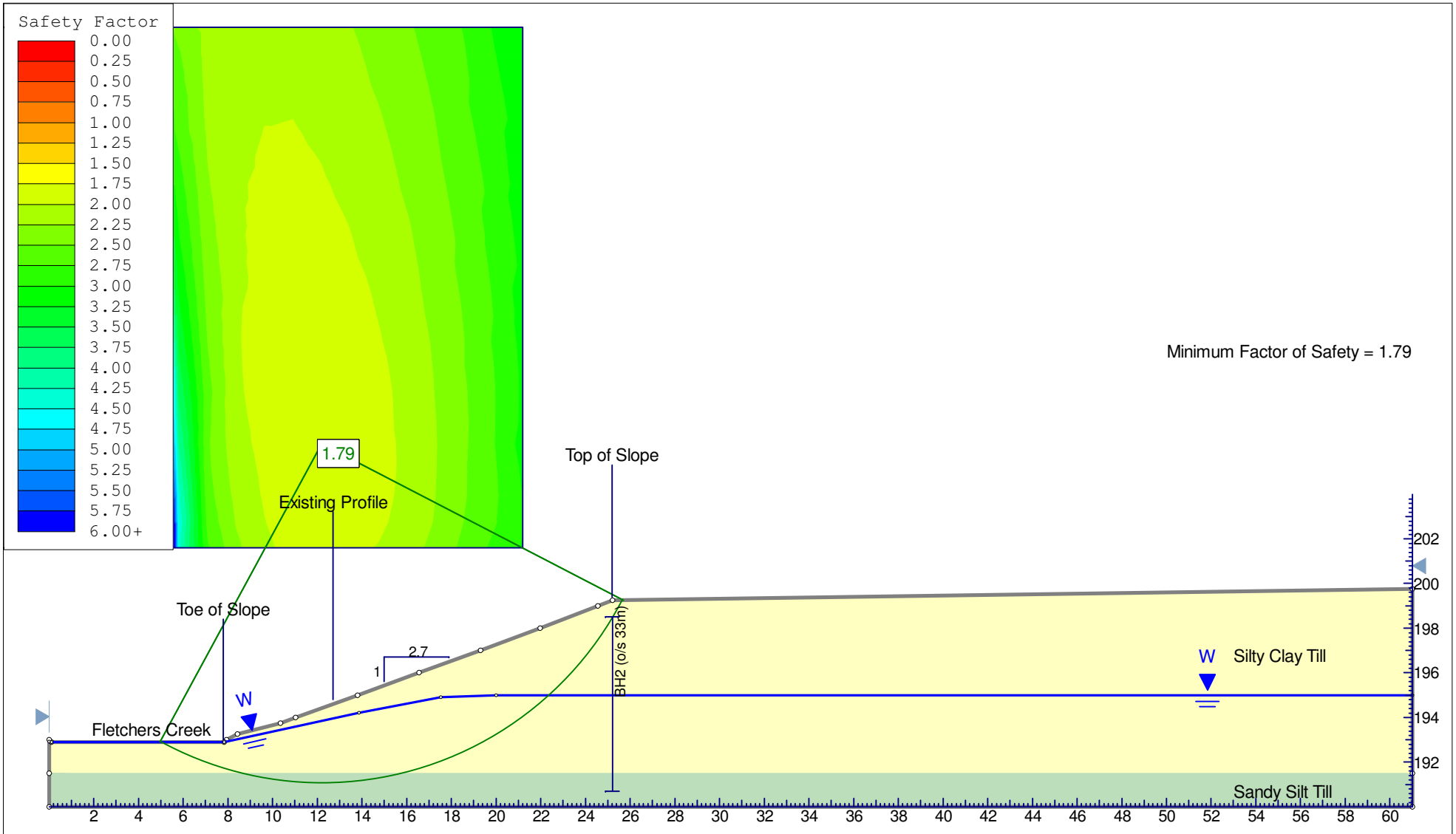
Project Title: Cross-Section A-A			Load Case: Stable Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: HWY	Checked By: BL	Scale: 1:250	Revision: 1
Date: April 2020		Reference No.: 0803-S002	Drawing No.: 3



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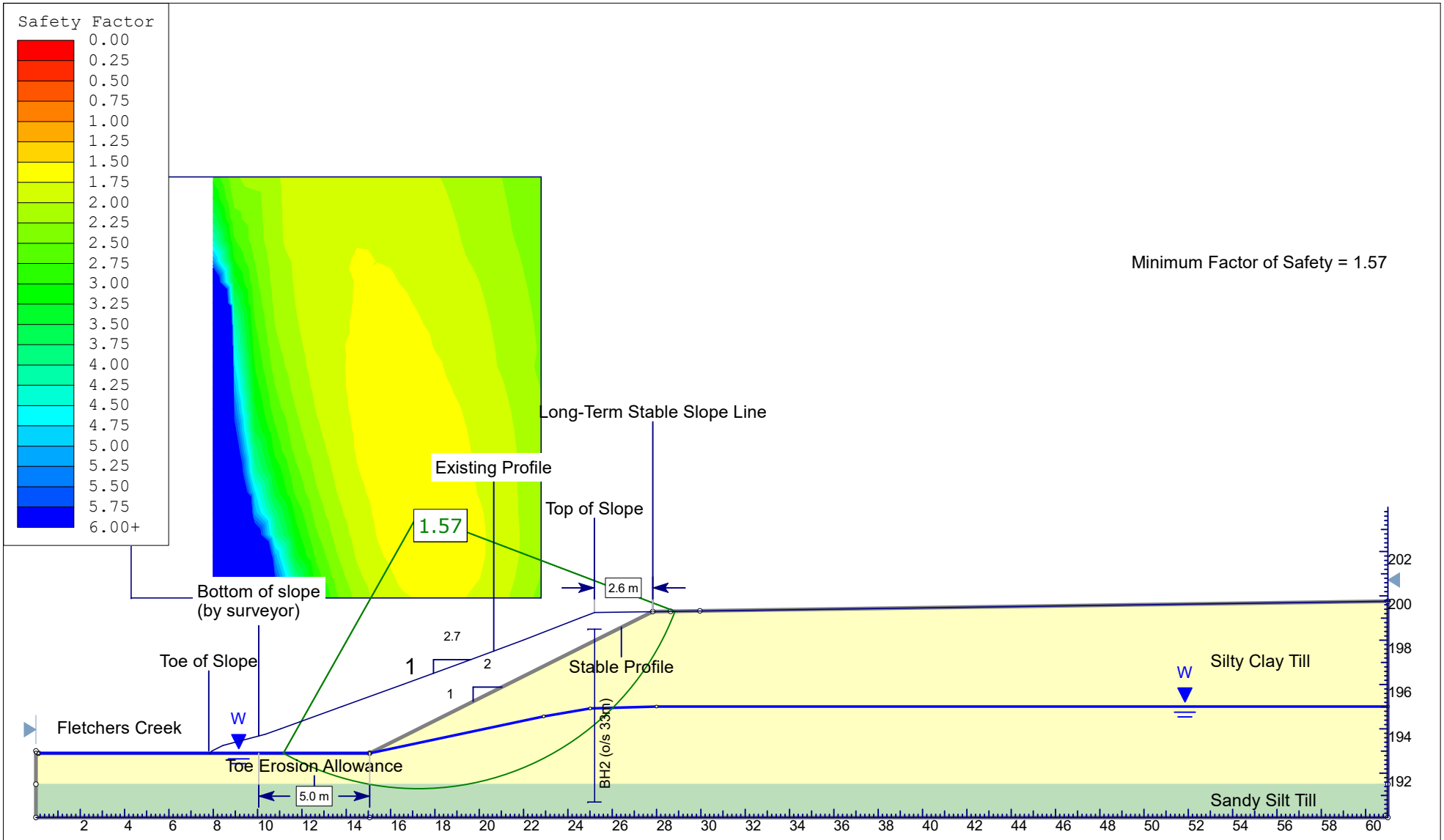
Project Title: Cross-Section B-B			Load Case: Existing Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: BL	Checked By:	Scale: 1:250	Revision: -
Date: January 2016		Reference No.: 0803-S002	Drawing No.: 4



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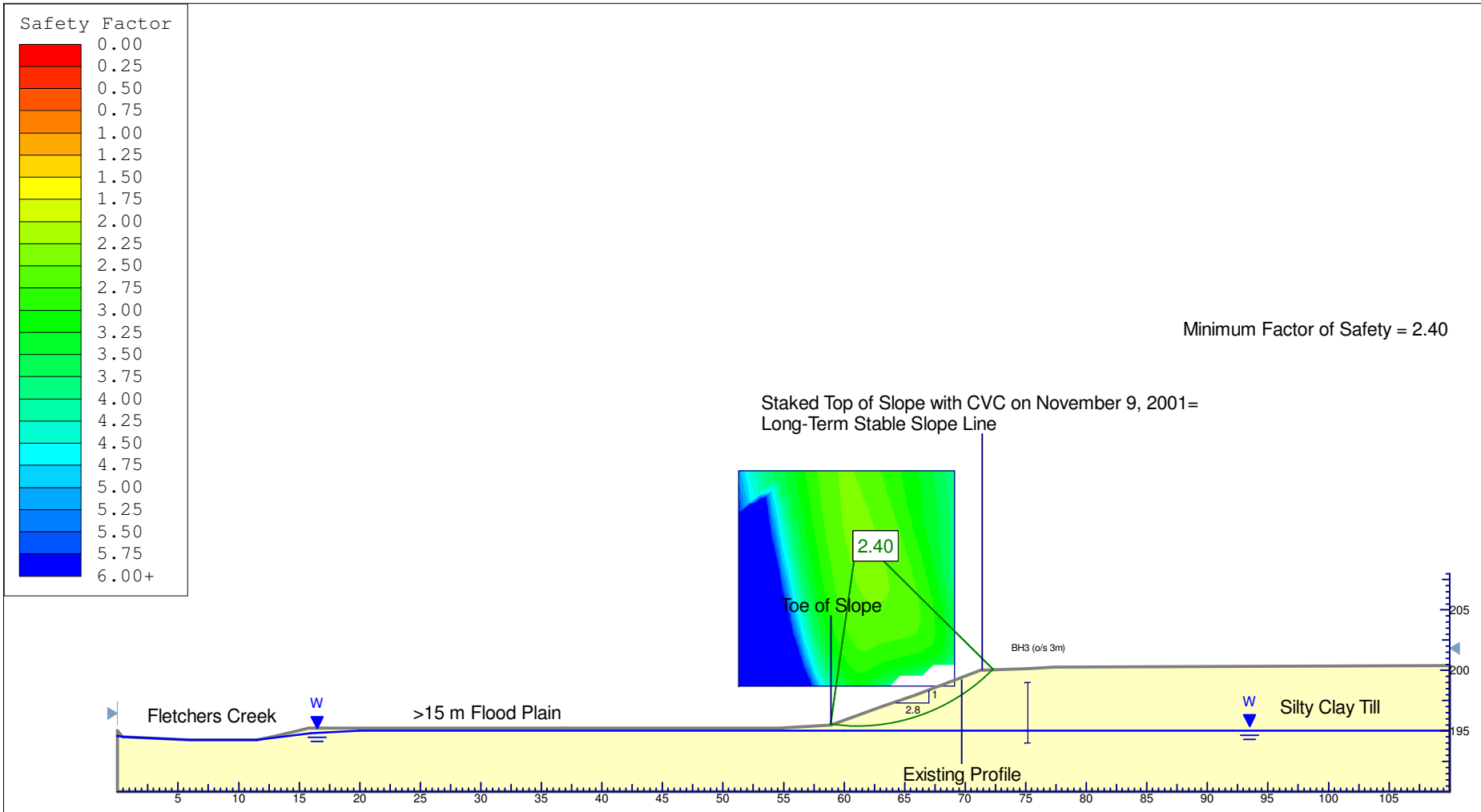
Project Title: Cross-Section B-B			Load Case: Stable Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By	Checked By: BL	Scale: 1:250	Revision: 1
Date: April 2020		Reference No.: 0803-S002	Drawing No.: 5



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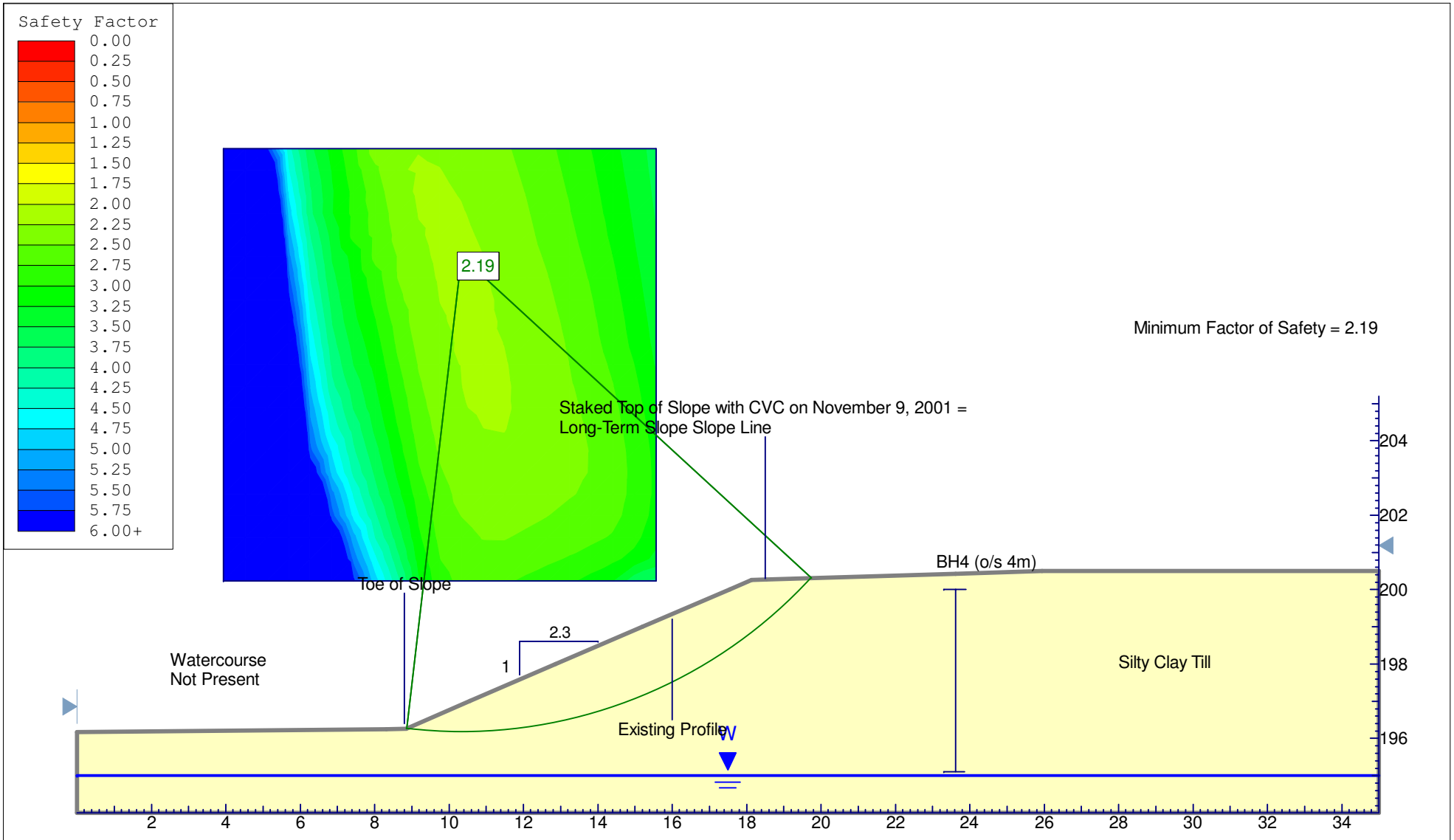
Project Title: Cross-Section C-C			Load Case: Existing Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: BL	Checked By:	Scale: 1:500	Revision: -
Date: January 2016		Reference No.: 0803-S002	Drawing No.: 6



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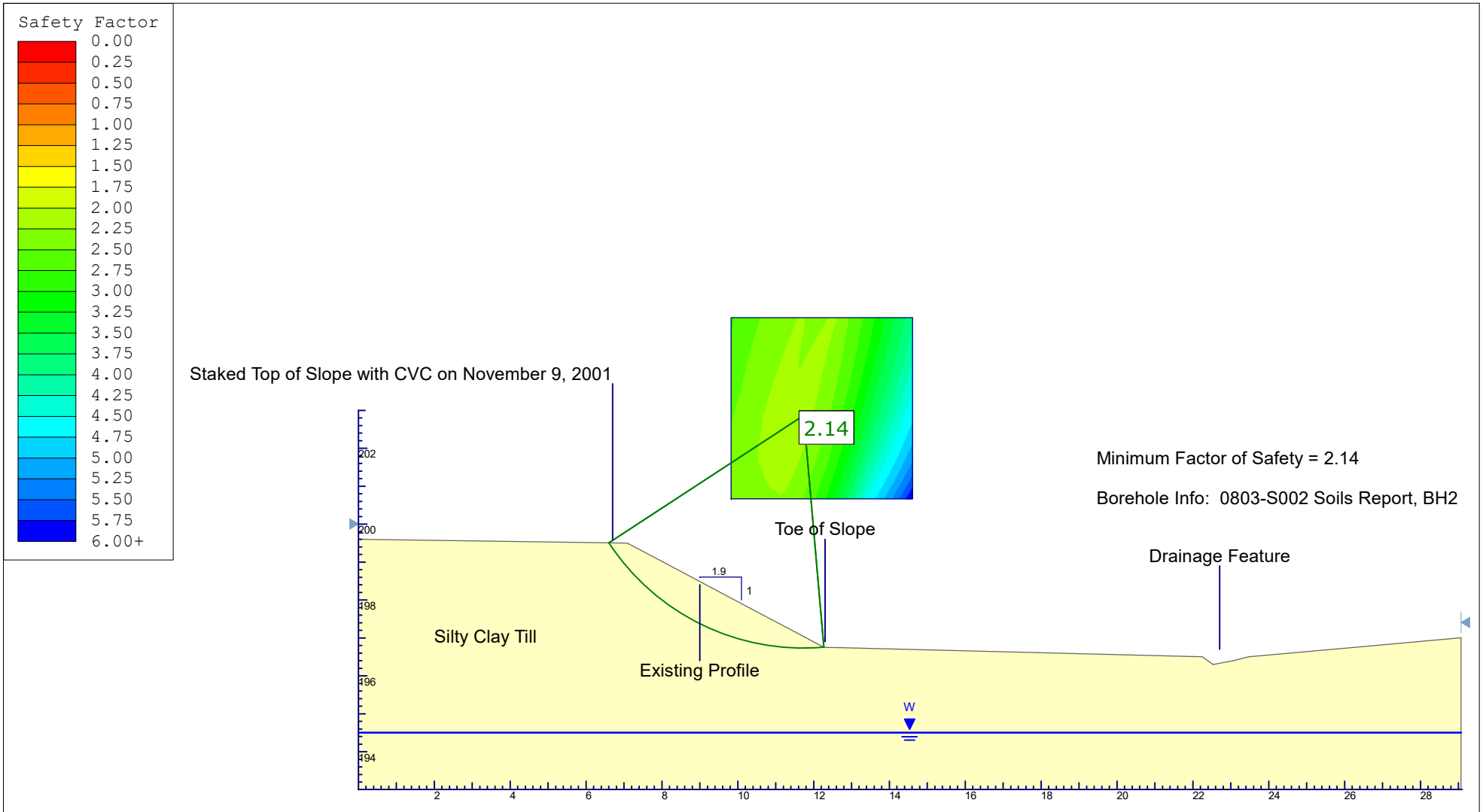
Project Title: Cross-Section D-D			Load Case: Existing Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: BL	Checked By:	Scale: 1:150	Revision: -
Date: January 2016		Reference No.: 0803-S002	Drawing No.: 7



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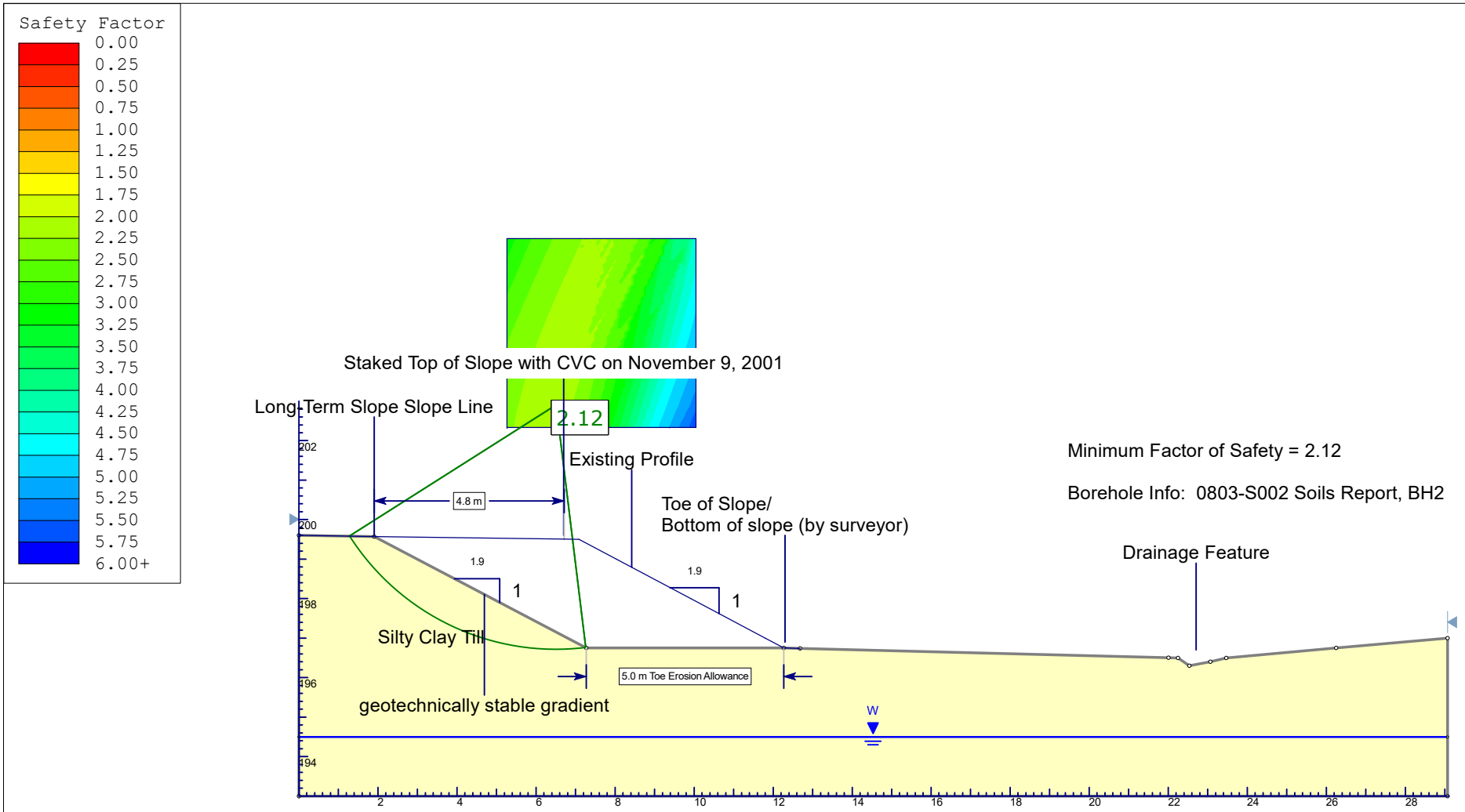
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

Project Title: Cross-Section E-E			Load Case: Existing Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: HWY	Checked By: BL	Scale: 1:150	Revision: 1
Date: April 2020		Reference No.: 0803-S002	Drawing No.: 8



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90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

Project Title: Cross-Section E-E			Load Case: Existing Condition (with Toe Erosion Allowance)
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: HWY	Checked By: BL	Scale: 1:150	Revision: -
Date: April 2020		Reference No.: 0803-S002	Drawing No.: 9

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90 WEST BEAVER CREEK ROAD, SUITE 100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	MUSKOKA	HAMILTON
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FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 721-7864	FAX: (905) 542-2769

A REPORT TO
DE ZEN REALTY COMPANY LIMITED

A GEOTECHNICAL INVESTIGATION FOR PROPOSED ROAD EXTENSION AND STORM OUTFALL

7140 HURONTARIO STREET

CITY OF MISSISSAUGA

REFERENCE NO. 2507-S026

SEPTEMBER 2025

DISTRIBUTION

Digital Copy - De Zen Realty Company Limited

TABLE OF CONTENT

TABLES

ENCLOSURES

1.0 INTRODUCTION

In accordance with an email authorization dated July 4, 2025, from Mr. Mark Palmieri of De Zen Realty Company Limited, a geotechnical investigation was carried out within a parcel of land known as 7140 Hurontario Street in the City of Mississauga.

The purpose of this investigation was to reveal the subsurface conditions and determine the engineering properties of the disclosed soils for the design and construction of a proposed Road Extension and Storm Outfall. The geotechnical findings and resulting recommendations are presented in this Report.

2.0 SITE AND PROJECT DESCRIPTION

The site is located in the Physiographic Region of Peel Plain, consisting of Bevelled Till Plains with clay to silt-textured till derived from glaciolacustrine deposits or shale.

The subject sites, currently known as 0 Vicksburgh Drive and 7174 Derrycrest Drive, have a total area of 17.6 hectares and is located at the south quadrant of Highway 407 and Hurontario Street in the City of Mississauga. The property is bounded by the Hydro One Easement to the north and northeast, Fletcher's Creek to the west, Derrydale Golf Course and an office building to the south, and Derrycrest Drive to the east. A portion of the property has been divided by a tributary of Fletcher's Creek. At the time of investigation, the properties are being used as farm land and a portion of the land along Derrycrest Drive was used as a winter light show venue which was being dismantled. The grading of the site gradually descends toward Fletcher's Creek to the west and south.

At the time of the report preparation, detailed design for the proposed development has not been finalized; however, it is understood that an Industrial Development is being proposed. As part of the development, a new road extension including a crossing over the tributary of Fletcher's Creek beyond the end of Vicksburgh Drive is proposed, along with a new storm outfall at the southwest limit of the property.

3.0 FIELD WORK

The field work, consisting of 3 sampled boreholes to depths ranging from 5.1 to 10.8 m, was performed on July 28 and August 1, 2025 at the locations shown on the Borehole Location Plan, Drawing No. 1, enclosed. The borehole locations were specified by Skira and Associates Ltd., the civil consultant of the project.

All boreholes were advanced at intervals to the sampling depths by a track-mounted machine with solid-stem augers for soil sampling. Standard Penetration Tests (SPTs), using the procedures described on the enclosed "List of Abbreviations and Terms", were performed at the sampling depths. The test results are recorded as the Standard Penetration Resistance (or 'N' values) of the subsoil. The compactness of the cohesionless strata and the consistency of the cohesive strata are inferred from the 'N' values. Split-spoon samples were recovered for soil classification and laboratory testing.

The field work was supervised and the findings were recorded by a Geotechnical Technician. The ground elevation at each of the borehole location was obtained using the Global Navigation Satellite System (GNSS).

4.0 SUBSURFACE CONDITIONS

The investigation revealed that beneath the topsoil veneer, and in the area fronting Vicksburgh Drive, a pavement structure and earth fill, the site is underlain by stratum of silty clay till and silty sand till.

Detailed descriptions of the encountered subsurface conditions are presented on the Boreholes Logs, comprising Figures 1, 2 and 3. The revealed stratigraphy is plotted on the Subsurface Profile, Drawing No. 2. The engineering properties of the disclosed soils are discussed herein.

4.1 Topsoil

The revealed topsoil is 8 cm to 20 cm in thickness encountered in Boreholes 2, 4 and 5 which are located at the vacant land.

4.2 Pavement Structure

A pavement structure, consisting of a layer of 100 mm thick asphalt and 510 mm thick granular fill, were encountered at Borehole 1, located at the cul-de-sac of Vicksburgh Gate.

4.3 Earth Fill

A layer of earth fill was encountered beneath the pavement structure at Borehole 1. The fill consists of sand and gravel. The earth fill extends to a depth of 1.5 m below the prevailing ground surface.

4.4 Silty Clay Till

The silty clay till was encountered in all boreholes. It extends to the borehole depths of 4.6 m to 7.6 m from the ground surface and is the predominant soil in the revealed stratigraphy. It consists of a random mixture of particle sizes ranging from clay to gravel, with the silt and clay being the dominant fraction. Sample examination indicates that it is sandy and contains a trace of gravel, with occasional sand seams, cobbles and boulders. Grain size analysis was performed on a silty clay till sample; the result is plotted on Figure 6.

The obtained 'N' values range from 7 to 54 blows, with a median of 25 blows per 30 cm of penetration, indicating the silty clay till is firm to hard, being generally very stiff in consistency. The surficial till is weathered, extending to depths of 0.6 m to 1.3 m below the prevailing ground surface.

The water content of silty clay till samples range between 10% and 18%, with a median of 13%, indicating moist to very moist, generally moist condition. Atterberg limits of a silty clay till sample is carried out, having Liquid Limit of 25% and Plastic Limit of 16%, showing that the sample is low in plasticity.

The engineering properties of the silty clay till deposit are presented below:

- High frost susceptibility and high soil-adfreezing potential.
- Low water erodibility.
- It will generally be stable in a relatively steep cut; however, prolonged exposure will allow the fissures in the weathered zone and the wet sand seams and layers to become saturated, which may lead to localized sloughing.

4.3 Silty Sand Till

The silty sand till was contacted beneath the silty clay till in Boreholes 2 and 3. It extends to the borehole depths of 6.6 m to 10.8 m from the ground surface. It consists of a random mixture of particle sizes ranging from clay to gravel, with the sand and silt being the dominant fraction. Sample examination indicates that it contains a trace of clay, some gravel and occasional sand seams. Grain size analysis was performed on 1 representative sample of the till and the result is plotted on Figure 7.

The compactness of the deposit is dense to very dense, being generally very dense, as inferred from the 'N' values ranging from 47 to over 100 blows, with a median over 100 blows per 30 cm of penetration.

The natural water content of the soil samples ranges from 8% to 10%, with a median of 9%, showing generally moist conditions.

The engineering properties of the till deposit are listed below:

- Moderate frost susceptibility.
- Moderate water erodibility.
- The till will be stable in relatively steep cuts; however, under prolonged exposure, localized sheet sliding may occur.

4.5 Compaction Characteristics of the Revealed Soils

The obtainable degree of compaction is primarily dependent on the soil moisture and, to a lesser extent, on the type of compactor used and the effort applied. As a general guide, the typical water content values of the revealed soils for Standard Proctor compaction are presented in Table 1.

Table 1 - Estimated Water Content for Compaction of On-Site Material

Soil Type	Determined Natural Water Content (%)	Water Content (%) for Standard Proctor Compaction	
		100% (optimum)	Range for 95% or +
Silty Clay Till	10 to 18 (median 13)	16	12 to 21
Silty Sand Till	8 to 10 (median 9)	10 to 12	6 to 17

* The above values are provided as a guideline. Standard Proctor Tests must be performed on bulk samples collected from site during construction prior to backfill and compaction.

5.0 GROUNDWATER CONDITION

All boreholes remained dry on completion of the field work.

The groundwater level will fluctuate with seasons and affected by the water level of Fletcher's Creek.

6.0 DISCUSSION AND RECOMMENDATIONS

The investigation revealed that beneath the topsoil veneer, and in localized area, pavement structure consisting of asphalt and granular fill overlying a layer of earth fill in the area fronting Vicksburgh Drive, the site is underlain by stratum of silty clay till and silty sand till.

At the time of the report preparation, design for the proposed development has not been finalized; however, it is understood a road extension will be provided beyond Vicksburg Gate for the future industrial development, including a road crossing over a tributary of Fletcher's Creek and a new storm outfall at the southwest limit of the property. The geotechnical findings warranting special consideration for the proposed project are presented below:

1. The topsoil must be removed for site development. It can only be re-used for landscaping in designated areas.
2. After demolition of the existing structures and foundations, the debris must be removed and disposed off-site.
3. The native soils are weathered extending to depths ranging from 0.6 to 1.3 m from the prevailing ground surface. It is weak and will consolidate under surcharge loads. To upgrade the weathered soils to engineered status, they must be subexcavated, sorted, aerated and properly compacted.
4. Due to the presence of earth fill, weathered soils, the subgrade of proposed structures must be inspected by a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to assess its suitability for bearing the designed foundations.
5. Further investigation will be necessary for the proposed industrial buildings and associated structures of the development.

The recommendations appropriate for the design of the development are presented herein. One must be aware that the subsurface conditions may vary between boreholes. Should subsurface variances become apparent during construction, a geotechnical engineer must be consulted.

6.1 **Road Embankment and Crossing Approach**

Where the grading is to be raised at the road crossing, the on-site native soils, where its moisture is properly controlled, is generally suitable for reuse for the embankment construction. The placement of the earth fill must be completed as engineered fill. The requirements for the construction of the road embankment and site grading by engineered fill are presented below:

The topsoil must be stripped and removed for development. It can be stockpiled on site and reused in landscaped areas.

The existing structures and foundations must be demolished and the debris must be removed and disposed off-site. The backfill must be free of topsoil or deleterious material, placed and compacted to engineered fill specifications.

1. The earth fill and weathered soils must be subexcavated, sorted free of topsoil inclusions or deleterious materials, if any, prior to its reuse as engineered fill.
2. The exposed subgrade must be inspected and proof-rolled prior to any fill placement.
3. Inorganic soils must be used for the fill, and they must be uniformly compacted in lifts 20 cm thick to 98% or + of the maximum Standard Proctor dry density (SPDD) up to the proposed pre-grade or finished grade. The soil moisture must be properly controlled near the optimum. If the foundations are to be built soon after the fill placement, the densification process for the engineered fill must be increased to 100% SPDD.
4. If the engineered fill is compacted with the moisture content on the wet side of the optimum, the underground services and pavement construction should not begin until the pore pressure within the fill mantle has completely dissipated. This must be further assessed at the time of the engineered fill construction.
5. If imported fill is to be used, it should be inorganic soils, free of any deleterious material with environmental issue (contamination). Any potential imported earth fill from off-site must be reviewed for geotechnical and environmental quality by the appropriate personnel as authorized by the developer or agency, before it is hauled to the site.
6. The engineered fill must not be placed during the period where freezing ambient temperatures occur either persistently or intermittently. This is to ensure that the fill is free of frozen soils, ice and snow.
7. Where the fill is to be placed on a bank steeper than 3 horizontal (H):1 vertical (V), the face of the bank must be flattened to 3H:1V or flatter so that it is suitable for safe operation of the compactor and the required compaction, as well as long-term stability of the slope can be obtained.
8. The fill operation must be supervised on a full time basis and monitored by a technician under the direction of a geotechnical engineer.
9. The engineered fill envelope and finished elevations must be clearly and accurately defined in the field, and they must be precisely documented.
10. Any excavation carried out in the certified engineered fill must be reported to the geotechnical consultant who supervised the fill placement in order to document the locations of excavation and/or to supervise reinstatement of the excavated areas to engineered fill status. If construction on the engineered fill does not commence within a period of 2 years from the date of certification, the condition of the engineered fill must be assessed for re-certification.
11. The footing and underground services subgrade must be inspected by the geotechnical consulting firm that supervised the engineered fill placement. This is to ensure that the foundations and service pipes are placed within the engineered fill envelope, and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation.

6.2 Road Crossing and Storm Outfall

At the time of the report preparation, detailed design for the proposed road crossing over the tributary of Fletcher's Creek and the Storm outfall has not been finalized. Preliminary recommendations for the design of the road crossing and storm outfall are presented below.

Road Crossing (Borehole 2)

Based on the borehole findings, the proposed road crossing can be constructed on footings founded below the weathered soils into the competent native soil or on engineered fill. The recommended bearing pressures for the design of the conventional footings, founded between El. 197.4 m and El. 195.4 m, are presented below:

- Maximum Bearing Pressure at Serviceability Limit State (SLS) = 150 kPa
- Factored Bearing Pressure at Ultimate Limit State (ULS) = 240 kPa

Due to the decreasing 'N' values with depth, limited bearing capacity can be provided for the road crossing at shallow depths. Where higher bearing capacity is required, caissons (drilled piers) can be considered. The recommended bearing pressures for the design of the drilled piers, founded below at a depth of 189 m, are presented below:

- Maximum Bearing Pressure at Serviceability Limit State (SLS) = 800 kPa
- Factored Bearing Pressure at Ultimate Limit State (ULS) = 1250 kPa

The drilling operation will require extra effort at lower depths due to the presence of very dense till, cobbles and boulders.

To facilitate ease of subgrade inspection and cleaning, a metal liner should be used to seal off any groundwater seepage or prevent loose soils from caving into the cavity. The caissons should not be less than 80 cm in diameter. The liner can be removed after the pier is filled with concrete.

Alternatively, Helical piers can be considered to support the proposed crossing. The appropriate founding elevation is expected to be at least 9 m or deeper below the ground surface. The design load of Helical Piers can be assessed by the prospective foundation contractor in these specialties. Full scale load test in the field must be conducted to confirm the load carrying characteristics of piers/piles.

The installation of the piers/piles must be supervised and inspected by either a geotechnical engineer, or a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the construction of piles is compatible with the foundation design requirements.

Additional review of the proposed crossing should be carried out once the detailed design becomes available.

Storm Outfall (Borehole 3)

Based on the borehole findings, the proposed storm outfall can be constructed on footings founded below the weathered soils into the competent native soil or on engineered fill. The recommended bearing pressures for the design of the conventional footings, founded below El. 198.5 m, are presented below:

- Maximum Bearing Pressure at Serviceability Limit State (SLS) = 200 kPa
- Factored Bearing Pressure at Ultimate Limit State (ULS) = 320 kPa

General Recommendations

The total and differential settlements of footings designed for the bearing pressure at SLS are estimated to be 25 mm and 20 mm, respectively.

During construction, the foundation subgrade must be inspected by a geotechnical engineer or a senior geotechnical technician to ensure that the revealed conditions are compatible with the foundation design requirements.

The foundation of the road crossing and outfall should extend into the sound native soils below the frost depth of 1.2 m or the scouring depth, whichever is deeper.

If groundwater seepage is encountered in excavation, the foundation must be poured immediately after subgrade inspection or the subgrade should be protected by a concrete mud-slab immediately after exposure. This will prevent construction disturbance and costly rectification of the bearing subsoil.

The building foundation should meet the requirements specified in the latest Ontario Building Code and the structures should be designed to resist an earthquake force using Site Classification ‘D’ (stiff soil).

6.3 Underground Services

The underground services should be founded on sound native soil or properly compacted inorganic earth fill. Where incompetent or weathered soil is encountered, it should be subexcavated and replaced with the bedding material, compacted to at least 98% SPDD.

A Class 'B' bedding is recommended for the underground services construction. It should consist of compacted 19-mm (3/4") Crusher-Run Limestone (CRL), or equivalent, as approved by a geotechnical engineer.

The pipe joints connecting into the manholes and catch basins must be leak-proof to prevent the migration of fines through the joints. Openings to subdrains and catch basins should be shielded with a fabric filter to prevent blockage by silting.

A soil cover of at least equal to the diameter of the pipe should be in place at all times after pipe installation, to prevent pipe floatation when the trench is deluged with water derived from precipitation.

The on-site clayey soils are considered moderately high in corrosivity to ductile iron pipes and metal fittings; therefore, the underground services should be protected against soil corrosion. For estimation for the anode weight requirements, the electrical resistivities disclosed in Table 4 can be used. The proposed anode weight must meet the minimum requirements as specified by the Region and Municipality Standard.

6.4 Backfilling in Trenches and Excavated Areas

The on site inorganic soils are suitable for use as trench backfill. Where the soils are either too dry or on the dry site of the optimum, the soil may require wetting prior to structural compaction. Where the soils are wet, they must be aerated by spreading thinly on the ground or mixed with drier soil prior to structure compaction. The weathered soils must be sorted free of concentrated topsoil and organics, if any, before reusing for structural backfill and/or engineered fill.

When compacting the till on the dry side of the optimum, the compactive energy will frequently bridge over the chunks in the soil and be transmitted laterally into the soil mantle. Therefore, the lifts should be limited to 20 cm or less (before compaction), or to a suitable thickness assessed by test strips performed by the compaction equipment. Boulders over 15 cm in size must be sorted and removed from the backfill.

The backfill in service trenches should be compacted to at least 95% SPDD, increasing to 98% SPDD below the concrete floor slab and within 1.0 m below the pavement. The material should be compacted with the water content at 2% to 3% drier than the optimum.

In normal construction practice, the problem areas of pavement settlement largely occur adjacent to manholes, catch basins, services crossings, foundation walls and columns. In confined areas where the desired slope cannot be achieved or the operation of a proper heavy-duty compactor cannot be facilitated, sand fill or granular backfill, which can be appropriately compacted by using a smaller vibratory compactor, should be used.

Road Embankment

Backfill around the new road crossing should consist of non-frost susceptible granular material. If a culvert is considered, the backfill must be placed and compacted simultaneous on all sides to prevent unbalanced loading on the culvert. It should be compacted to at least 98% SPDD in 20 cm lifts, or to a suitable thickness as assessed by test strips performed by the equipment which will be used at the time of construction.

Earth fill of the road embankment beyond the culvert crossing can consist of selected on-site or imported inorganic soils, compacted uniformly to 98% SPDD in 20 cm lifts.

The proposed embankment should be graded at a 3 Horizontal (H):1 Vertical (V) or flatter for stability and sodded/vegetated to protect against rainwash erosion. Where steeper slope gradient is considered, a separate stability assessment must be performed to verify the stability of the slope.

The compaction must be inspected by a geotechnical technician under the supervision of a geotechnical engineer, to ensure that the backfill is compatible for road construction.

6.5 Pavement Design

The proposed road extension may be a private condominium road or a local industrial road. The pavement designs are presented in Table 2.

Table 2 - Pavement Design (Private Road and Local Industrial Road)

Course	Thickness (mm)	OPS Specifications
Asphalt Surface	40	HL3
Asphalt Binder Private Road Local Industrial Road	 65 100	 HL8 HDBC
Granular Base	200	Granular 'A'
Granular Sub-base Private Road Local Industrial Road	 250 400	Granular 'B', Type I

The final subgrade must be proof-rolled using a heavy roller or loaded dump truck. Any soft spot as identified must be rectified by subexcavation and replacing with selected dry inorganic material. The subgrade within 1.0 m below the underside of the granular sub-base must be compacted to at least 98% SPDD, with the water content at 2% to 3% drier than its optimum. All the granular bases should be compacted in 150 to 200 mm lifts to 100% SPDD.

The pavement subgrade will suffer a strength regression if water is allowed to saturate the mantle. The following measures should, therefore, be incorporated in the construction procedures and road design:

- If the road is to be constructed during the wet seasons or the subgrade is unstable, the top 1.0 m of the subgrade should be replaced with drier, compacted, selected subgrade material or the top 0.8 m of the subgrade should be replaced with granular material. This can be determined at the time of road construction.
- The subgrade should be properly crowned and smooth-rolled to allow interim precipitation to be properly drained prior to pavement construction.
- Lot areas adjacent to the roads should be properly graded to prevent ponding of large amounts of water. Otherwise, the water will seep into the subgrade mantle and induce a regression of the subgrade strength, with costly consequences for the pavement construction.
- Fabric filter-encased curb subdrains connecting to a positive outlet of catch basin, will be required on both sides of the roadway.

6.6 **Soil Parameters**

The recommended soil parameters for the project design are given in Table 3.

Table 3 - Soil Parameters

Unit Weight and Bulk Factor	**Unit Weight γ (kN/m³)**		**Estimated Bulk Factor**	
	Bulk	**Submerged**	**Bulk**	**Submerged**
Silty Sand Till	22.0	12.5	1.33	1.03
Silty Clay Till	21.5	12.5	1.30	1.05
Lateral Earth Pressure Coefficients		**Active K_a**	**At Rest K_o**	**Passive K_p**
Silty Sand Till		0.30	0.40	3.33
Silty Clay Till		0.35	0.45	2.86
Coefficient of Permeability (K) and Percolation Time (T)				
		K (cm/sec)		**T (min/cm)**
Silty Sand Till		10^{-7}		80+
Silty Clay Till		10^{-4}		12
Effective Shear Strength Parameters				
		Cohesion c' (kPa)		**Angle of Internal Friction, ϕ'**
Silty Sand Till		2		31°
Silty Clay Till		5		30°
Estimated Electrical Resistivity				
Silty Clay Till				3000 ohm·cm
Silty Sand Till				5000 ohm·cm
Coefficients of Friction				
Between Concrete and Granular Base				0.50
Between Concrete and Sound Native Soils				0.35

6.7 Excavation

Excavation should be carried out in accordance with Ontario Regulation 213/91. For excavation purposes, the types of soils are classified in Table 4.

Table 4 - Classification of Soils for Excavation

Material	Type
Sound Tills	2
Earth Fill and Weathered soils	3

In the tills, any perched groundwater yield can be collected and removed by conventional pumping from sumps.

The hard and very dense tills contain cobbles and boulders. Extra effort and a properly equipped backhoe will be required for excavation. Boulders and shale fragments larger than 15 cm in size are not suitable for structural backfill.

Prospective contractors must be asked to assess the in situ subsurface conditions for soil cuts by digging test pits to 1.0 m below the anticipated depth of excavation. These test pits should be allowed to remain open for a few hours to assess the trenching conditions.

7.0 LIMITATIONS OF REPORT

This report was prepared by Soil Engineers Ltd. for the account of De Zen Realty Company Limited for review by their designated consultants and government agencies. The material in this report reflects the judgement of Sze Wing Yu, B.Eng., and Kelvin Hung, P.Eng., in light of the information available to it at the time of preparation.

Use of this report is subject to the conditions and limitations of the contractual agreement. Any use which a Third Party makes of this report, or any reliance on decisions to be made based on it, is the responsibility of such Third Parties. Soil Engineers Ltd. accepts no responsibility for damages, if any, suffered by any Third Party as a result of decisions made or actions based on this report.

SOIL ENGINEERS LTD.

Sze Wing Yu, B.Eng.
SY/KH

Kelvin Hung, P.Eng.

LICENSED PROFESSIONAL ENGINEER
9/23/25
K. W. K. HUNG
100124295
PROVINCE OF ONTARIO

LIST OF ABBREVIATIONS AND DESCRIPTION OF TERMS

The abbreviations and terms commonly employed on the borehole logs and figures, and in the text of the report, are as follows:

SAMPLE TYPES

AS Auger sample
CS Chunk sample
DO Drive open (split spoon)
DS Denison type sample
FS Foil sample
RC Rock core (with size and percentage recovery)
ST Slotted tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance or 'N' Value:

The number of blows of a 63.5 kg hammer falling from a height of 76 cm required to advance a 51 mm outer diameter drive open sampler 30 cm into undisturbed soil, after an initial penetration of 15 cm.

Plotted as '○'

Dynamic Cone Penetration Resistance:

A continuous profile showing the number of blows per each 30 cm of penetration of a 51 mm diameter, 90° point cone driven by a 63.5 kg hammer falling from a height of 76 cm.

Plotted as '—●—'

WH Sampler advanced by static weight
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
NP No penetration

SOIL DESCRIPTION

Cohesionless Soils:

'N' (blows/30 cm)	Compactness
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
> 50	very dense

Cohesive Soils:

Undrained Shear Strength (kPa)	'N' (blows/30 cm)	Consistency
<12	<2	very soft
12 to <25	2 to <4	soft
25 to <50	4 to <8	firm
50 to <100	8 to <15	stiff
100 to 200	15 to 30	very stiff
>200	>30	hard

Method of Determination of Undrained Shear Strength of Cohesive Soils:

x 0.0 Field vane test in borehole; the number denotes the sensitivity to remoulding
△ Laboratory vane test

METRIC CONVERSION FACTORS

1 ft = 0.3048 m
1 inch = 25.4 mm
1 lb = 0.454 kg
1 ksf = 47.88 kPa

JOB NO.: 2507-S026

LOG OF BOREHOLE: 1

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Road Extension and Storm Outfall

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 7140 Hurontario Street, City of Missisauga

DRILLING DATE: July 28, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
202.2	Pavement Surface							
0.0	**100 mm ASPHALT** **510 mm GRANULAR**	1	DO	34	0		4	
	Brown **EARTH FILL** sand and gravel	2	DO	12	1		11	
200.7 1.5	Very stiff to hard **SILTY CLAY TILL** sandy a trace of gravel occ. sand seams	3	DO	15	2		11	
		4	DO	21			13	
		5	DO	38	3		11	
	brown grey	6	DO	16	4 5		12	Dry on completion
195.6		7	DO	16	6		10	
6.6	**END OF BOREHOLE**				7 8 9 10 11 12			

Soil Engineers Ltd.

JOB NO.: 2507-S026

LOG OF BOREHOLE: 2

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Road Extension and Storm Outfall

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 7140 Hurontario Street, City of Missisauga

DRILLING DATE: July 28, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
198.4	Ground Surface							
0.0	**20 cm TOPSOIL**	1A	DO	7	0		16	Dry on completion
	Firm to hard	1B						
	SILTY CLAY TILL weathered							
	sandy							
	a trace of gravel							
	occ. sand seams	2	DO	25	1		12	
		3	DO	32	2		13	
		4	DO	33			11	
	brown grey				3			
		5	DO	44			10	
					4			
		6	DO	17	5		11	
					6			
		7	DO	8			17	
190.8					7			
7.6	Grey, very dense	8	DO	50/13	8		8	
	SILTY SAND TILL							
	a trace of clay							
	some gravel							
		9	DO	53/15	9		9	
					10			
187.6		10	DO	50/13			9	
10.8	**END OF BOREHOLE**				11			
					12			

Soil Engineers Ltd.

JOB NO.: 2507-S026

LOG OF BOREHOLE: 3

FIGURE NO.: 3

PROJECT DESCRIPTION: Proposed Road Extension and Storm Outfall

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 7140 Hurontario Street, City of Missisauga

DRILLING DATE: August 1, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
199.8	Ground Surface							
0.0	Brown, stiff to hard **SILTY CLAY TILL** sandy a trace of gravel occ. sand seams, coubles and boulders weathered	1	DO	8	0		18	
		2	DO	19	1		15	
		3	DO	20	2		13	
		4	DO	34	3		12	
		5	DO	45	4		13	
195.2								
4.6	Grey, dense to very dense **SILTY SAND TILL** a trace of clay some gravel occ. sand seams	6	DO	47	5		9	
		7	DO	55	6		10	
193.2								
6.6	**END OF BOREHOLE**				7			
					8			
					9			
					10			
					11			
					12			

Dry on completion

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

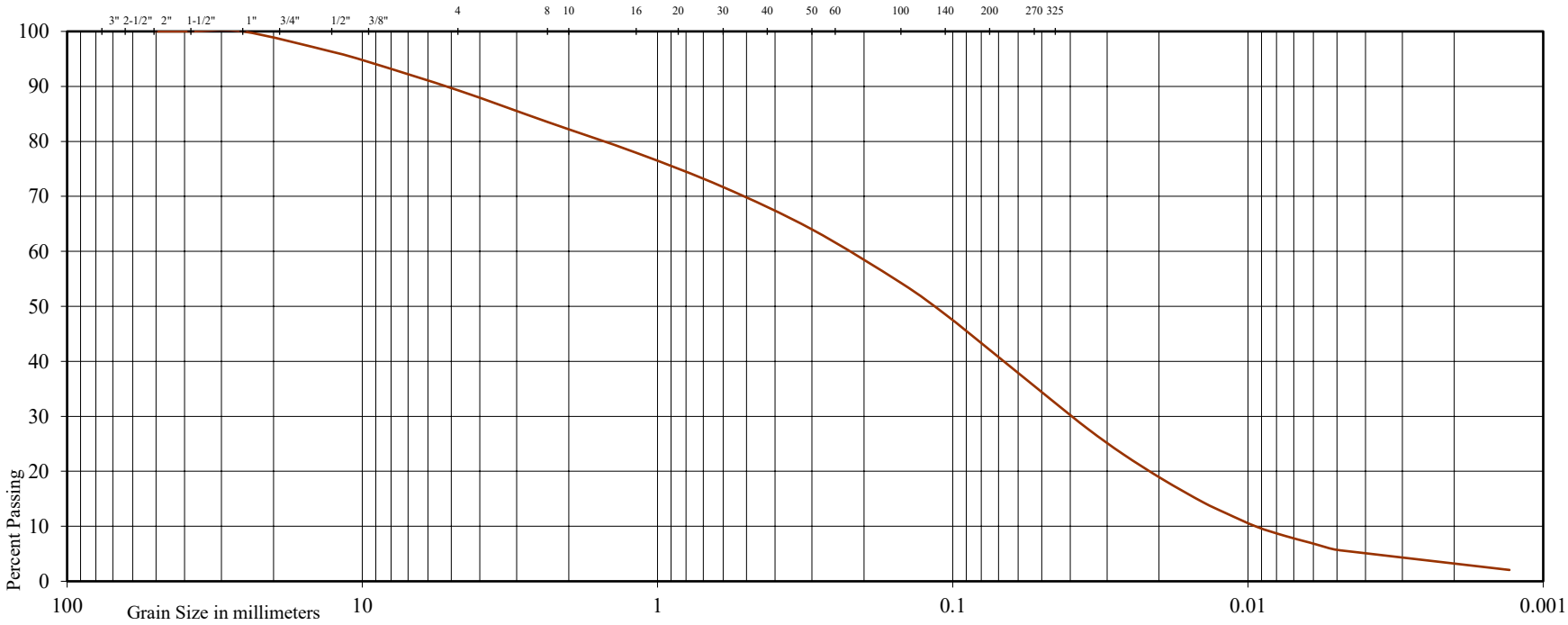
Reference No: 2507-S026

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Road Extension and Storm Outfall

Location: 7140 Hurontario Street, City of Missisauga

Borehole No: 2

Sample No: 8

Depth (m): 7.9

Elevation (m): 190.5

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = 8

Estimated Permeability (cm./sec.) = 10^{-4}

Classification of Sample [& Group Symbol]: SILTY SAND TILL

a trace of clay, some gravel

Figure: 4

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

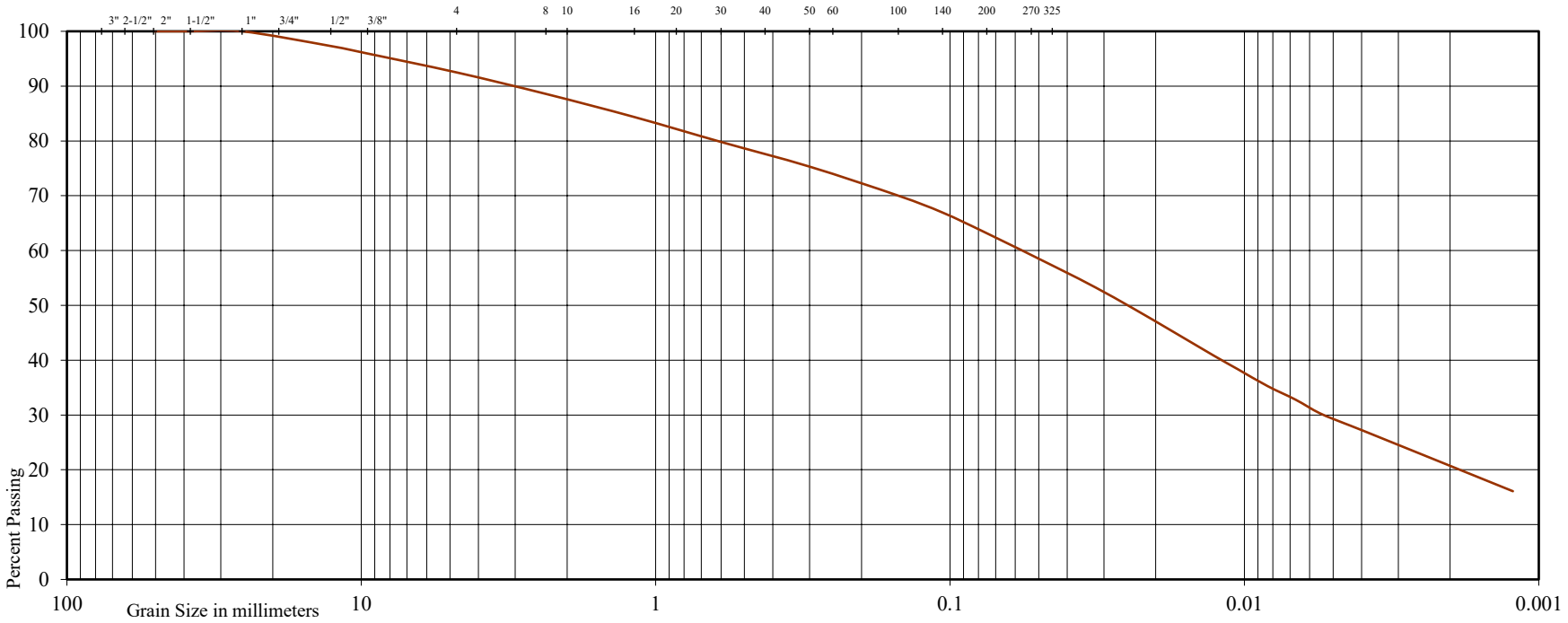
Reference No: 2507-S026

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Road Extension and Storm Outfall

Location: 7140 Hurontario Street, City of Missisauga

Borehole No: 3

Sample No: 5

Depth (m): 3.4

Elevation (m): 196.4

Liquid Limit (%) = 25

Plastic Limit (%) = 16

Plasticity Index (%) = 9

Moisture Content (%) = 13

Estimated Permeability (cm./sec.) = 10^{-7}

Classification of Sample [& Group Symbol]: SILTY CLAY TILL
sandy, a trace of gravel

Figure: 5

LEGEND

Borehole location

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

BOREHOLE LOCATION PLAN

SITE: 7140 HURONTARIO STREET, CITY OF MISSISSAUGA

DESIGNED BY: -	**CHECKED BY:** -	**DWG NO.:** 1	
SCALE: -	**REF. NO.:** 2507-S026	**DATE:** September 2025	**REV** 1:125

Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE

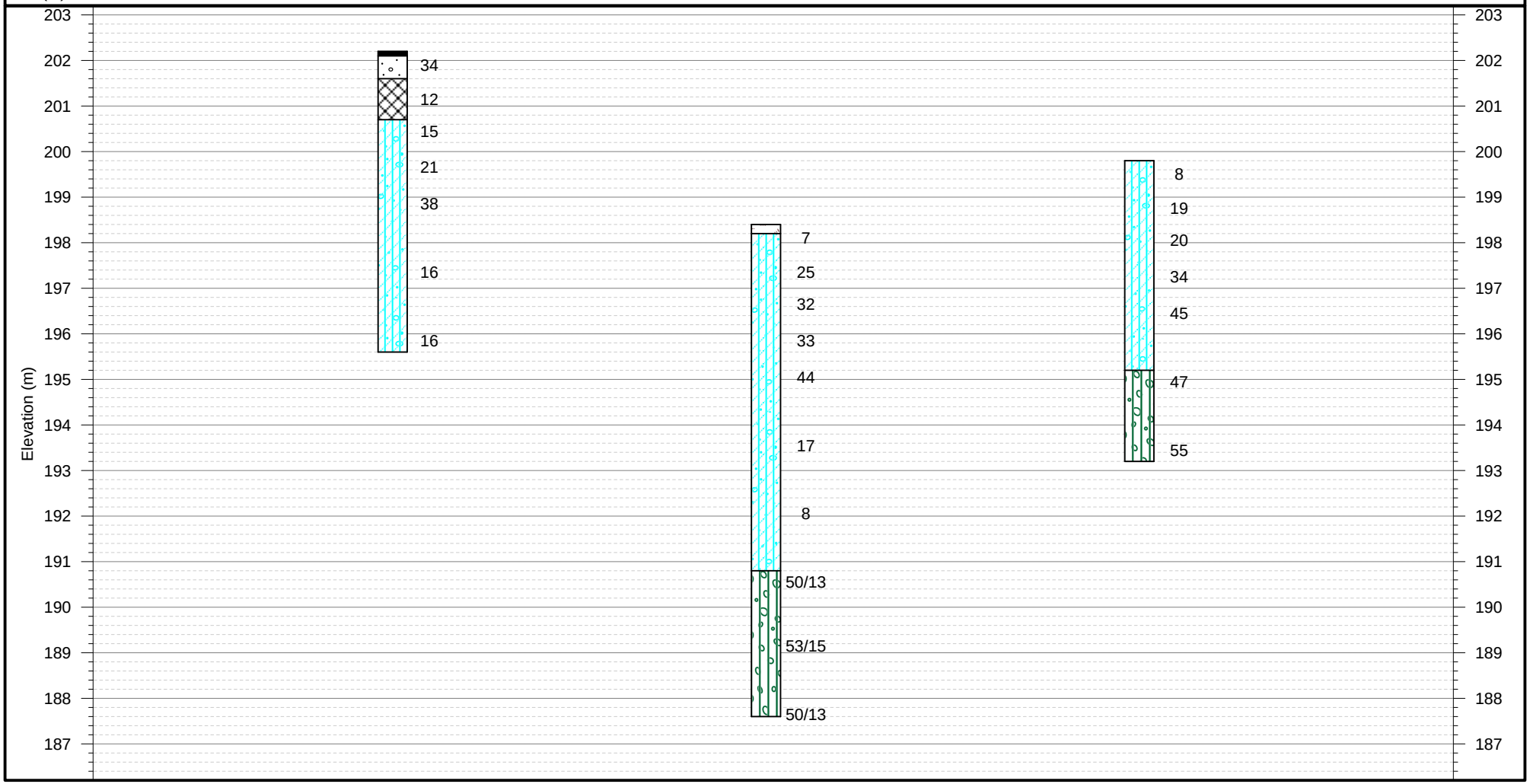
SUBSURFACE PROFILE
DRAWING NO. 2
SCALE: AS SHOWN

JOB NO.: 2507-S026
REPORT DATE: September 2025
PROJECT DESCRIPTION: Proposed Road Extension and Storm Outfall

PROJECT LOCATION: 7140 Hurontario Street, City of Missisauga

LEGEND

TOPSOIL | GRANULAR | SILTY SAND TILL | SILTY CLAY TILL
ASPHALT | FILL



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BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	MUSKOKA	HAMILTON
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FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 721-7864	FAX: (905) 542-2769

September 9, 2025

De Zen Realty Company Limited
4890 Tomken Road, Units 1-4
Mississauga, Ontario
L4W 1J8

Attention: Mr. Mark Palmieri

Re: Monitoring Well Installation, Groundwater Monitoring and In-Situ Guelph Permeameter Infiltration Testing Report
Proposed Road Extension
7140 Hurontario Street
City of Mississauga

Dear Sir:

As requested, Soil Engineers Ltd. (SEL) has completed a monitoring well installation and groundwater monitoring program along with in-situ infiltration testing and laboratory analysis for the collected shallow soil samples in support of the proposed Low Impact Development (LID) stormwater management infrastructure planning and design at the captioned development site located at 7140 Hurontario Street, in the City of Mississauga, Ontario (the Subject Site). The assessment and findings are presented in the current letter report.

1.0 Introduction

The Subject Site is located at the south quadrant of the overpass of Highway 407 and Hurontario Street, in the City of Mississauga. The Subject Site is currently known as 0 Vicksburg Drive and 7174 Derrycrest Drive. As per the reviewed plans shared to SEL, the Subject Site covers an area of 17.60 hectares (1,76,000.0 m^2). The Subject Site is bounded by the Hydro One Easement to the north and northeast, Fletcher's Creek to the west, Derrydale Golf Course and an office building to the south, and Derrycrest Drive, a vacant land and a commercial property to the east. A portion of the Subject Site has been divided by a tributary of Fletcher's Creek. At the time of investigation, the west portion of the Subject Site was being used as farm land and a portion of the land along Derrycrest Drive was used as a winter light show venue which was being dismantled. The grading of the Subject Site gradually descends toward Fletcher's Creek to the west and south. The Site Location Plan can be reviewed on **Drawing No. 1**.

2.0 Borehole Advancement and Monitoring Well Installation

Drilling boreholes and construction of monitoring wells were conducted by SEL within the vicinity of the proposed stormwater management tanks as requested by the client on August 01, 2025. The program consisted of drilling two (2) boreholes (BH) extending to a depth 4.6 metres below ground surface (mbgs) and installing monitoring wells within each BH. The location of the boreholes and monitoring wells is shown on **Drawing No. 2**.

Borehole drilling and monitoring well construction were completed by a licensed water well contractor, under the full-time supervision of SEL's geotechnical supervisor who logged the soil strata encountered during borehole advancement and collected representative soil samples for textural classification. The boreholes were drilled using a track-mounted drill rig equipped with solid stem augers and split spoons. Detailed descriptions of the encountered subsoil and groundwater conditions are provided by SEL and presented on the borehole and monitoring well logs, in the enclosed **Appendix A**.

The monitoring wells were constructed using 50-mm diameter PVC pipes at the two (2) borehole locations. 1.5 m long 10-slot well screens were installed at two (2) monitoring well locations. All monitoring wells were equipped with monument steel casing at the ground surface.

The UTM coordinates and ground surface elevations at the monitoring wells' locations, as well as the monitoring well construction details, are presented in **Table 1**. The ground surface elevations and horizontal coordinates at the monitoring well locations were determined at the time of the investigation, using the Trimble TSC3 handheld Global Navigation Satellite System.

Table 1 - Monitoring Well Installation Details

Monitoring Well ID	Installation Date	UTM Coordinates (m)		Ground El. (masl)	Screen Interval (mbgs)	Soil in the Screen Interval	Casing Dia. (mm)	Protective Casing Type
		Easting	Northing					
BH/MW 4	August 01, 2025	603473	4833519	200.50	3.1 – 4.6	Silty Clay Till	50	Monument
BH/MW 5	August 01, 2025	603593	4833346	200.00	3.1 – 4.6	Silty Clay Till	50	Monument

Note:
mbgs: metres below ground surface
masl: metres above sea level

3.0 Subsoil Lithology

The study has disclosed beneath the layer of topsoil, 8 to 13 cm thick, the Subject Site is generally underlain by a stratum of silty clay till to a maximum termination depth of investigation at 4.6 metres below ground surface (mbgs). Subsoil profile contacted at the BH/MW locations is presented below:

Silty Clay Till: Beneath the layer of topsoil, a silty clay till layer was encountered at both BH/MW locations. The silty clay till unit extended completely to the termination depth of investigation i.e. 4.6 mbgs. It consists of a random mixture of particle sizes ranging from clay to gravel, with the silt and clay being the dominant fraction. Sample examination indicates that it is sandy and contains a trace of gravel, with occasional sand seams, cobbles and boulders. The unit was found to be stiff to hard in relative consistency. For BH/MW 4, it was found to be brown in color to a depth of 3.0 mbgs and changed to grey below 3.0 mbgs, whereas, for BH/MW 5, it was found to be brown to the termination depth of investigation at 4.6 mbgs.

Detailed description of subsoil lithology is provided in **Appendix A (Figure 1** to **2)**, inclusive.

4.0 Groundwater Monitoring Program

The groundwater levels in the monitoring wells were monitored, manually over three (3) monitoring events on August 05 and 18, 2025 and September 02, 2025, to record the fluctuation of the shallow groundwater table beneath the Subject Site. SEL measured the groundwater levels using an interface probe (Heron Water Tape Series #1900).

A summary of the groundwater level observations and their corresponding elevations are provided in **Table 2**.

Table 2- A Summary of Groundwater Monitoring

MW ID	Unit	Groundwater Level		
		Aug 05, 2025	Aug 18, 2025	Sep 02, 2025
BH/MW 4	mbgs	> 4.60	4.40	4.10
	masl	< 195.90	196.10	196.40
BH/MW 5	mbgs	>4.60	4.30	3.80
	masl	<195.40	195.70	196.20

Notes:
mbgs metres below ground surface
masl metres above sea level

5.0 In-situ Infiltration Tests

The study was undertaken to verify the permeability and percolation times of the subsoil profile at the proposed location and the depth of the proposed LID measures in the Subject Site. The study will confirm the feasibility of subsoil for the design of infiltration infrastructure to manage collected stormwater runoff, for its redirection to the subsurface, to recharge groundwater, and to meet the stormwater management planning and design objectives for the proposed developed site.

5.1 Work Program

SEL intended to conduct in-situ percolation testing at the proposed locations and depths provided by Skira & Associates Ltd. The testing was designed to be conducted at the approximate base of the proposed LID infiltration facilities and ± 0.5 m below the proposed base. This testing will more accurately determine the local infiltration rate and will assist in appropriately sizing the facility.

Our representative performed the site visit on August 05, 2025, to conduct the in-situ infiltration (percolation) testing. An excavator was used to create pits in order to achieve the target depths at the test locations, in the vicinity for the proposed LID infrastructures. The in-situ infiltration (percolation) testing program was conducted at two (2) test locations. The approximate test locations, where the in-situ percolation tests were attempted are shown on **Drawing 2**, enclosed. The ground surface elevations at the test pit locations, are provided in **Table 3**.

Table 3 - Test Locations and Depths

Test Location	UTM Coordinates (m)		Ground Surface Elevation (masl)	Depth (mbgs)	Estimated Bottom Elevation (masl)
	Easting	Northing			
1	603568	4833324	± 200.1	± 3.2	± 196.9
				± 3.7	± 196.4
2	603453	4833498	± 200.5	± 4.2	± 196.3
				± 4.7	± 195.8

Note:
mbgs: metres below ground surface
masl: metres above sea level

For this study, four (4) in-situ infiltration tests were conducted within two (2) test pit excavations; one (1) test each at the bottom of elevation for the proposed LID infrastructure, and one (1) test each at a depth of ±0.5 meters below the bottom elevations for the proposed LID infrastructure. The in-situ infiltration percolation testing was performed using the Guelph Permeameter (GP) instrument (Model

2800K1). The in-situ tests were conducted at the approximate depth, ranging from ±3.2 m to ±4.7 m below the existing ground surface (mbgs) at elevations, ranging from ±195.8 to 196.9 metres above sea level (masl).

5.2 Methodology

The Guelph Permeameter (GP) is a constant head infiltrometer instrument which operates using the Marriott Bottle principle, whereby a constant ponded head of water is allowed to infiltrate into the unsaturated surface soil via a small test hole augured to penetrate the shallow soil horizon. The constant water head for the instrument is maintained within the test hole by an air tube and its height setting within the instrument. A steady-state flow of water into the subsoil should result after a relatively short period, corresponding to the instrument's ponded water head setting. A higher ponded water head setting should result in a higher steady-state flow of water into the subsoil, than the previous, lower ponded water head setting.

The Field Saturated (fs) hydraulic conductivity (K) estimates for the soil are obtained using the mathematical equations established for the instrument, whereby two (2) steady-state flow measurements, corresponding to the two (2) ponded water head settings for the instrument, can be used as minimum information to estimate the field saturated hydraulic conductivity (K_{fs}) for the soil. The K_{fs} estimates can also be determined, using the one-ponded water head approach which corresponds to one steady-state water flow measurement into the subsoil.

It is critical to note that the K_{fs} and infiltration rates are two (2) different concepts and that translation from one parameter to another cannot be done through unit a of conversion. In accordance with the guidelines from the Toronto and Region Conservation Authority (TRCA) and Credit Valley Conservation Authority (CVC), the infiltration rates are based on the "Low Impact Development Stormwater Management Planning and Design Guide, Table C1", as provided in **Table 4**, below.

Table 4 - Approximate Relationship between K_{fs}, Percolation Time and Infiltration Rate

Hydraulic Conductivity, K_{fs} (cm/sec)	Percolation Time, T (min/cm)	Infiltration Rate, 1/T (mm/hr)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

K_{fs} – field saturated hydraulic conductivity
Source: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997; Supplementary Guidelines to the Ontario Building Code 1997, SG-6 Percolation Time and Soil Description.

5.3 Infiltration Test Results and Discussion

The in-situ infiltration tests were conducted at depths, ranging from ± 3.2 to ± 4.7 mbgs. Based on the visual observations in the field, the existing shallow subsoils beneath the site comprise:

- Silty Clay Till with trace of gravel

The results from the attempted in-situ infiltration testing and review and interpretation of the soil grain size analyses for the collected sub-soil samples obtained at the testing depths are summarized in **Table 5**.

Table 5 - Estimated Hydraulic Conductivity, Percolation Times, and Infiltration Rates

Test Location	Test and Depth (m)	Soil Type	Ponded Water Head Setting (cm)	Steady State Flow (cm^3/sec)	Grain Size Permeability Estimation (cm/sec)	Guelph Permeameter Method	In-situ testing of Hydraulic Conductivity (cm/sec)	Estimated Infiltration Rate (mm/hr)
1	**A** ± 3.7	Silty Clay Till, sandy, a trace of gravel	(H1) 10	N/A	10^{-7} (Appendix B, Figure 1) (GS)	N/A	N/A	N/A
			(H2) 22	N/A				
	***B** ± 3.2	Silty Clay Till with trace of gravel	(H1) 10	N/A	-	N/A	N/A	N/A
			(H2) 22	N/A				
2	***A** ± 4.7	Silty Clay Till with trace of gravel	(H1) 10	N/A	-	N/A	N/A	N/A
			(H2) 20	N/A				
	B ± 4.2	Silty Clay Till, sandy, a trace of gravel	(H1) 11	N/A	10^{-7} (Appendix B, Figure 2) (GS)	N/A	N/A	N/A
			(H2) 22	N/A				

GS – Soil Grain Size Distribution
N/A – The test was not successful due to low permeable soil
** Inferred soil textured based on visual examination*

Attempts were made to complete the in-situ infiltration tests. However, due to the presence of compact, low-permeable shallow subsoils at both the testing locations, the in-situ percolation test could not be successfully performed using the GP instrument at a depth of 3.2 and 3.7 mbgs at Test Location 1, and at a depth of 4.2 and 4.7 mbgs at Test Location 2.

Since the use of Guelph Permeameter was not successful at providing results at the testing locations, at the indicted depths, representative subsoil samples were also collected at the testing depths for follow-up, laboratory, grain size analysis as per Ministry of Transpiration (MTO) Laboratory Standards 602 and 702 to confirm the shallow subsoil sample texture classifications in order to interpret and estimate the soil's percolation (T) times and associated infiltration rates. The results for the soil grain size analyses for the collected soil samples, as performed in our laboratory are enclosed for your reference, **Appendix B** (**Figure 1** to **2**), inclusive.

Based on the interpretation from the soil grain size analyses plots, as performed in our laboratory, the estimated permeability for the subsoils contacted at the testing locations (1A and 2B) is found to be 1.0×10^{-7} cm/sec. The estimated infiltration rates for the subsoil contacted at the testing depths (1A and 2B), is found to be approximately 7.3 mm/hr. The corresponding estimated Percolation T-Time for 1A and 2B is 80 min/cm. The infiltration rates will vary depending on the soil density and the amounts of sand and/or silt present in the native subsoils, and for the presence of any vertical soil structure fracturing, and/or macro pores.

5.4 General Comments

Toronto and Region Conservation Authority (TRCA) design manual, titled "Low Impact Development Stormwater Management Planning and Design Guide", a safety factor is to be incorporated in the determining of the infiltration rates for the design of LID infiltration infrastructure, such as infiltration galleries, soak away pits or similar technology that would be implemented as part of stormwater management planning and design for site development.

Test Location 1: The attempt to complete the in-situ infiltration test at the test location 1A and 1B was unsuccessful due to the occurrence of low-permeability subsoil at the indicated depth for 1A and 1B. As such, the in-situ percolation tests cannot be interpreted at this test location.

Test Location 2: The attempt to complete the in-situ infiltration test at the test location 2A and 2B was unsuccessful due to the occurrence of low-permeability subsoil at the indicated depth for 2A and 2B. As such, the in-situ percolation tests cannot be interpreted at this test location.

It is a common practice to maintain infiltration elevations at least 1.0 m above the highest groundwater level. The shallow groundwater table was monitored on three (3) events, and the findings can be reviewed in **Section 4.0** of the current letter report.

Soil exhibiting percolation rate less than of 15 mm/hr does not meet the minimum 15 mm/hr infiltration rate required for conventional designs for LID infrastructure, such as infiltration galleries, soak away pits, or similar technology that would be implemented as part of stormwater management planning and design for the proposed site development. Other methods such as thickening of topsoil within landscaped areas should be also considered to meet the LID planning and design objectives for stormwater management design throughout portions of the proposed development site as an alternative means for addressing LID infrastructure.

Any infiltration system infrastructure designed for the Subject Site should include an overflow valve to divert any excess runoff to a second infiltration gallery, or to a grass swale, or to the municipal storm sewer at the surface, should a high intensity runoff event not be adequately accommodated by any proposed holding tank, and/or an infiltration trench/exfiltration tank, or infiltration gallery completed for the developed site. Due to the low permeability for shallow native soils, limited opportunities exist to implement LIDs to promote infiltration and groundwater recharge at the developed site to address future stormwater management planning for the propose industrial development.

6.0 Closure

We trust that this correspondence addresses your current needs and ask that you contact the undersigned should you have any questions or require additional information.

Yours truly,
SOIL ENGINEERS LTD.

Gurkaranbir Singh, M.Eng., EIT
GS/NA

Narjes Alijani, M.Sc., P.Geo.

ENCLOSURES

Site Location Plan	Drawing No. 1
Borehole and Monitoring Well and Test Pit Location Plan	Drawing No. 2
Borehole and Monitoring Well Logs.....	Appendix A
Grain Size Distribution Graphs	Appendix B

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DRAWINGS

REFERENCE NO. 2507-W026

602500 603000 603500 604000 604500

4834000 4833500 4833000

RAVENSWOOD DRIVE
CHERRYTREE DRIVE
ESDALE CIRCLE
MEADOWLARK DRIVE
TIMBERLANE DRIVE
HIGHWAY 407
TOPFLIGHT DRIVE
STATESMAN DRIVE
EXPORT BOULEVARD
EDWARDS BOULEVARD
HURONTARIO STREET
DERRY ROAD EAST
KINGSWAY DRIVE
DERRYCREST DRIVE
MCLAUGHLIN ROAD
MEADOWRIDGE COURT
PANHELLENIC DRIVE
WRIGLEY COURT
SAINT BARBARA BOULEVARD
COMISKEY CRESCENT
A Tributary of Fletchers Creek
Fletchers Creek
ROSSELENI DRIVE
MATISSE PLACE
WARHOL WAY
TWAIN AVENUE
VILLAGE WALK
ACUMEN COURT
DERRYDALE DRIVE
DERRY ROAD WEST
MARITZ DRIVE
LONGSIDE DRIVE

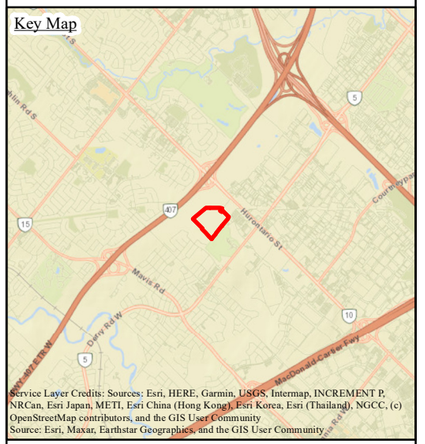
D:\Projects\2507-W026\

602500 603000 603500 604000 604500

N

References: Ontario Ministry of Natural Resources and Forestry
© King's Printer for Ontario, 2025

Key Map



Legend

- Approximate Boundary Of Subject Site
- Expressway/Freeway
- Major Road
- Local Road
- Waterbody
- Watercourse

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Site Location Plan

Hydrogeological Assessment
Proposed Road Extension and Storm Outfall
7140 Hurontario Street
City of Mississauga

Reference No. 2507-W026

Date: September 08, 2025

Scale:

0 37.5 75 150 225 300 375
Metres

Drawing No. 1

603200 603400 603600 603800

4833600 4833400 4833200

DERRY CREST DRIVE

BH/MW 4

TP 2

BH/MW 5

TP 1

Fletchers Creek

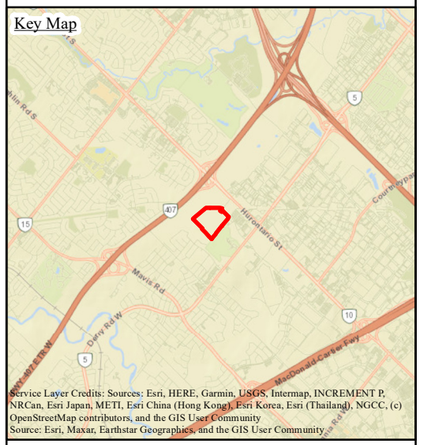
D:\Projects\2507-W026\

603200 603400 603600 603800

N

References: Ontario Ministry of Natural Resources and Forestry

Key Map



Service Layer Credits: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

Legend

- Approximate Boundary Of Subject Site
- Local Road
- Watercourse
- Borehole with Monitoring Well
- Test Pit

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Borehole and Monitoring Well and Test Pit Location Plan

Hydrogeological Assessment
Proposed Road Extension and Storm Outfall
7140 Hurontario Street
City of Mississauga

Reference No. 2507-W026

Date: September 08, 2025

Scale:
0 12.5 25 50 75 100 125
Metres

Drawing No. 2

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APPENDIX 'A'

BOREHOLE AND MONITORING WELL LOGS

REFERENCE NO. 2507-W026

JOB NO.: 2507-W026

LOG OF BOREHOLE:BH/MW 4

FIGURE NO.: 1

PROJECT DESCRIPTION: Proposed Road Extension

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 7140 Hurontario Street, City of Missisauga

DRILLING DATE: August 1, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
200.5	Ground Surface							
0.0	**13 cm TOPSOIL**	1A	DO	8	0		16	
	stiff to hard **SILTY CLAY TILL** sandy a trace of gravel occ. sand seams, coubles and boulders weathered	1B						
		2	DO	26	1		14	
		3	DO	17	2		15	
		4	DO	28			16	
	brown grey	5	DO	52	3		16	
195.9					4			
4.6	**END OF BOREHOLE**				5			
	Installed 50 mm Ø monitoring well to 4.6 m completed with 1.5 m screen Sand backfill from 2.4 to 4.6 m Bentonite seal from 0.0 m to 2.4 m Provided with a momument steel casing				6			
					7			
					8			
					9			
					10			
					11			
					12			

Water level reading:
Dry on Aug 05, 2025
W.L. @ El. 196.1 masl on Aug 18, 2025
W.L. @ El. 196.4 masl on Sep 02, 2025

Soil Engineers Ltd.

JOB NO.: 2507-W026

LOG OF BOREHOLE:BH/MW 5

FIGURE NO.: 2

PROJECT DESCRIPTION: Proposed Road Extension

METHOD OF BORING: Solid Stem Augers

PROJECT LOCATION: 7140 Hurontario Street, City of Missisauga

DRILLING DATE: August 1, 2025

El. (m) / Depth (m)	SOIL DESCRIPTION	SAMPLES Number	Type	N-Value	Depth Scale (m)	Dynamic Cone (blows/30 cm) 10 30 50 70 90 / Shear Strength (kN/m²) 50 100 150 200 / Penetration Resistance (blows/30 cm) 10 30 50 70 90	Atterberg Limits PL LL / Moisture Content (%) 10 20 30 40	WATER LEVEL
200.0	Ground Surface							
0.0	**8 cm TOPSOIL**	1A	DO	9	0		13	
	Brown, stiff to hard	1B						
	SILTY CLAY TILL							
	sandy							
	a trace of gravel							
	occ. sand seams, coubles and boulders	2	DO	26	1		13	
	weathered							
		3	DO	28	2		13	
		4	DO	41	3		14	
		5	DO	54			13	
195.4					4			
4.6	**END OF BOREHOLE**				5			
	Installed 50 mm Ø monitoring well to 4.6 m completed with 1.5 m screen Sand backfill from 2.4 to 4.6 m Bentonite seal from 0.0 m to 2.4 m Provided with a momument steel casing				6			
					7			
					8			
					9			
					10			
					11			
	Water level reading: Dry on Aug 05, 2025 W.L. @ El. 195.7 masl on Aug 18, 2025 W.L. @ El. 196.2 masl on Sep 02, 2025				12			

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APPENDIX 'B'

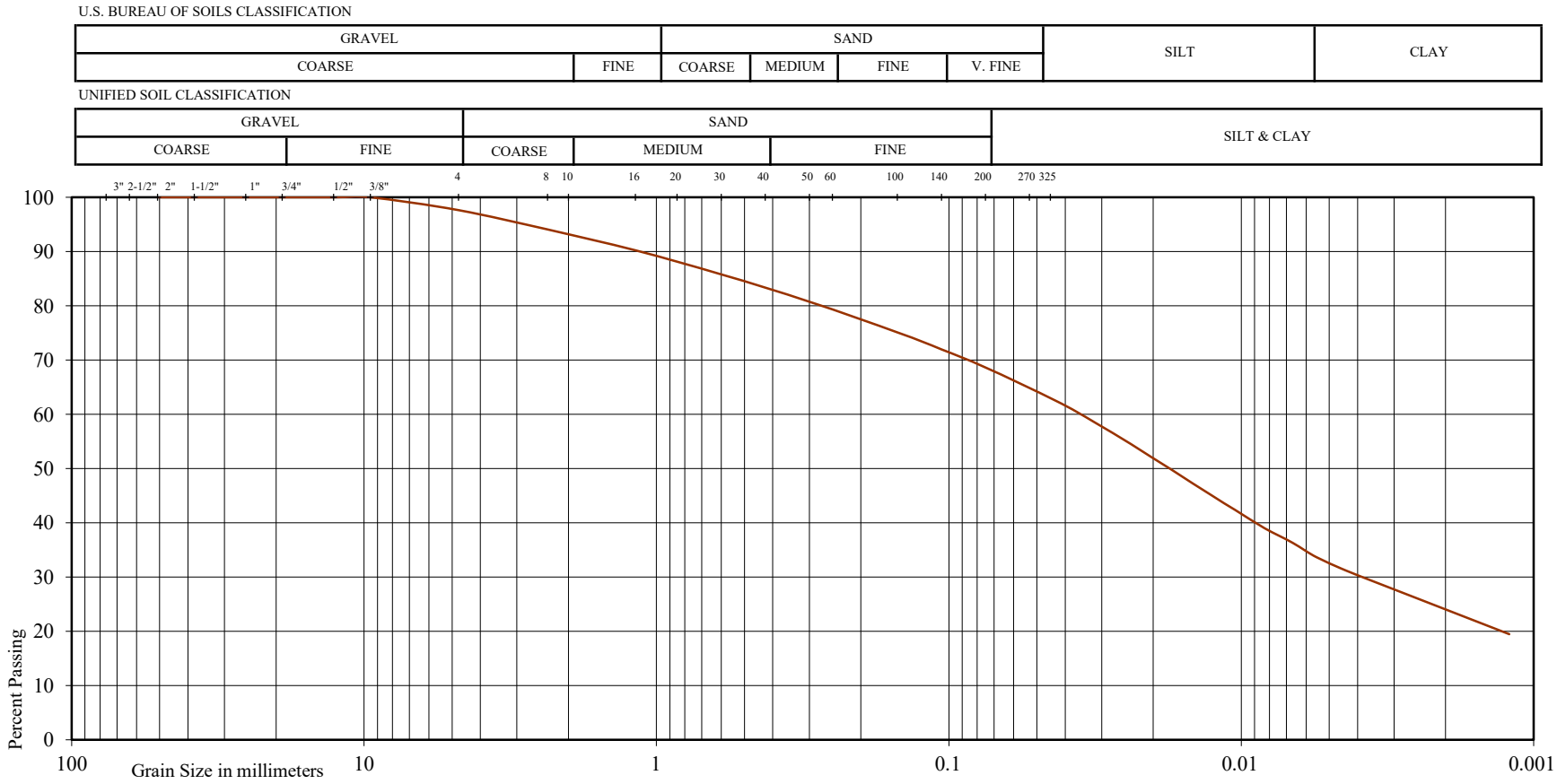
GRAIN SIZE DISTRIBUTION GRAPHS

REFERENCE NO. 2507-W026

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

Reference No: 2507-W026



Project: Proposed Road Extension

Location: 7140 Hurontario Street, City of Mississauga

Test Pit No: 1

Sample No: 1A

Depth (m): 3.7

Elevation (m):

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = -

Estimated Permeability (cm./sec.) = 10^{-7}

Estimated Percolation Time (min/cm) = 80

Classification of Sample [& Group Symbol]: SILTY CLAY, TILL
sandy, a trace of gravel

Figure: 1

Soil Engineers Ltd.

GRAIN SIZE DISTRIBUTION

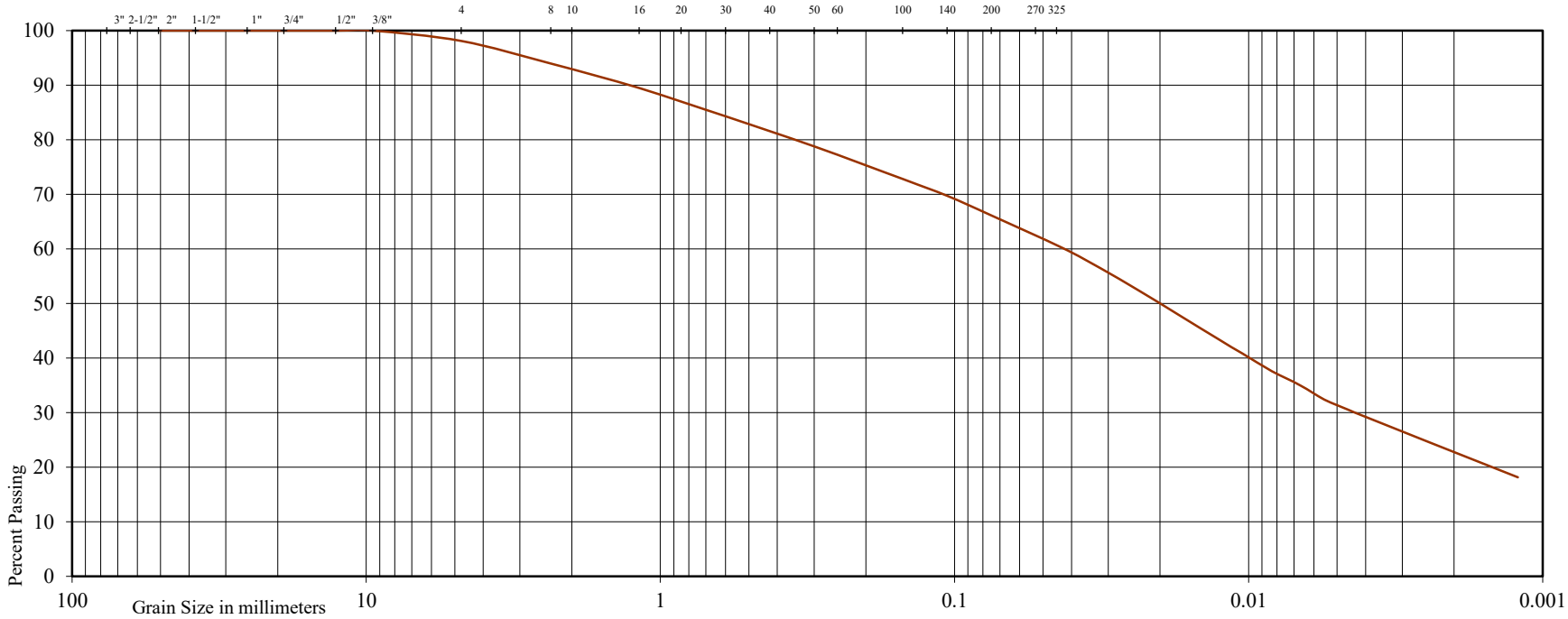
Reference No: 2507-W026

U.S. BUREAU OF SOILS CLASSIFICATION

GRAVEL		SAND				SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	V. FINE		

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND			SILT & CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Project: Proposed Road Extension

Location: 7140 Hurontario Street, City of Mississauga

Test Pit No: 2

Sample No: 2B

Depth (m): 4.2

Elevation (m):

Liquid Limit (%) = -

Plastic Limit (%) = -

Plasticity Index (%) = -

Moisture Content (%) = -

Estimated Permeability (cm./sec.) = 10^{-7}

Estimated Percolation Time (min/cm) = 80

Classification of Sample [& Group Symbol]: SILTY CLAY, TILL
sandy, a trace of gravel

Figure: 2

APPENDIX B

STORMWATER DESIGN SHEETS
PRE-DEVELOPMENT SUB-CATCHMENT SUMMARY
EXTERNAL FLOW VISUAL OTTHYMO RESULTS
CULVERT SIZING – CULVERT MASTER CALCULATIONS
GEOMORPHIC ASSESSMENT BY: GEOMORPHIX

SUBDIVISION : **DEZEN CONSTRUCTION COMPANY LTD**

MAJOR DRAINAGE **FLETCHER'S CREEK**

CITY FILE:

CONSULTANT : **SKIRA & ASSOCIATES LTD.**

CITY OF MISSISSAUGA

STORM SEWER DESIGN CHART

FINAL SUBMISSION

SHEET No. 1 of 1

PROJECT No. : 220-M10

DESIGNED BY : M.J.

DATE : SEP. 2025

$I_{(10YR)} = 1010/(Tc+4.6)^{0.78}$

MANNING'S ROUGHNESS COEFF. n = 0.013

LOCATION	FROM MH	TO MH	AREA Aa	RUNOFF COEFF. Ca	AaxCa	ACCUM. AREA A=∑Aa	ACCUM. AaxCa C=∑AaxCa	Tc	INTENSITY I	EXPECTED FLOW $Q=\frac{I \cdot A \cdot C}{360}$	TYPE OF PIPE	LENGTH L	SLOPE S	PIPE SIZE NOMINAL D	CAPACITY n=0.013 Q	VELOCITY n=0.013 V	TIME OF FLOW $T=\frac{L}{V \times 60}$	VELOCITY n = 0.009	VELOCITY ACTUAL	INVERT ELEV. UPPER	INVERT ELEV. LOWER
	MH#	MH#	ha			ha		min	mm/hr	m^3/s		m	%	mm	m^3/s	m/s	min	m/s		MH	MH
VIKCKSBURGH	EX.10	3	1.15	0.75	0.86	1.15	0.86	15.00	99.17	**0.237**	CONC	20.8	1.11	600	**0.676**	2.31	0.15	3.34			
	EX.6	3	3.58	0.75	2.69	3.58	2.69	15.00	99.17	**0.741**	CONC	20.5	2.34	600	**0.982**	3.36	0.10	4.85			
	3	2	0.28	0.90	0.25	5.01	3.80	15.15	98.58	**1.041**	CONC	77.2	2.12	675	**1.278**	3.46	0.37	4.99			
	EX.9	2	1.19	0.25	0.30	1.19	0.30	15.00	99.17	**0.083**	CONC	20.5	2.34	600	**0.982**	3.36	0.10	4.85			
	2	1	0.00	0.90	0.00	6.20	4.10	15.52	97.16	**1.107**	CONC	16.2	1.97	675	**1.232**	3.33	0.08	4.81			
PHASE 1	PLUG	1	3.85	0.90	3.47	3.85	3.47	15.00	140.69	**1.356**	CONC	37.0	0.56	1050	**2.133**	2.39	0.26	3.45	100 YR		
DERRYCREST	1	6	1.73	0.75	1.30	11.78	8.87	15.60	137.47	**3.387**	CONC	167.8	0.45	1500	**4.947**	2.71	1.03	3.92	100 YR		
PHASE 2	PH 2	HW	7.75	0.90	6.98	7.75	6.98	15.00	99.17	**1.923**	CONC	65.0	0.80	975	**2.093**	2.71	0.40	3.92			

Drainage Area Summary

Project Name: DeZen Industrial Lands
Municipality: City of Mississauga
Project No.: 220-M10
Date: 13-Apr-20

Prepared by: MJ
Checked by: MJ
Last Revised: 13-Apr-20

Catchment Area Summary

Area ID	Area	$C_{initial}$	$C(100YR)_{Adjusted}$	%IMP	CN	CN (AMCIII)	IA	Slope	TP
	(ha)						(mm)	(%)	(hr)
Ext. 1	2.55	0.90	1.00	1.00	77	89	2	0.47	0.38
Ext. 2	4.18	0.25	0.31	0.07	65	81	8	1.27	0.33
Ext. 3	0.88	0.50	0.63	0.43	71	85	5	1.38	0.22
Ext. 4	0.87	0.25	0.31	0.07	78	89	8	1.06	0.29
Ext. 5	1.80	0.25	0.31	0.07	75	88	8	1.22	0.28
Ext. 6	1.78	0.25	0.31	0.07	75	88	8	1.16	0.14
1	2.10	0.25	0.31	0.16	77	89	8	0.84	0.32
2	1.19	0.25	0.31	0.16	77	89	8	2.41	0.10
3	1.87	0.25	0.31	0.16	78	89	8	1.40	0.19
4	2.80	0.25	0.31	0.16	77	88	8	1.57	0.20
5	1.81	0.25	0.31	0.16	74	87	9	7.97	0.05

```
2
*****************************************************************************
*                                                                           *
*                     SKIRA & ASSOCIATES LIMITED                            *
*                           MARCH   2020                                    *
*                    DEZEN INDUSTRIAL   SUBDIVISION                         *
*                         PRE DEVELOPMENT                                   *
*                 STORMWATER MANAGEMENT PRACTICES                           *
*                                                                           *
*****************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                               *
*  100 YEAR DESIGN STORMS                                                   *
*****************************************************************************

START               TIME=0 METOUT=0 NSTORM=1 NRUN=1
                    100MIS4.CHI

READ STORM          STORM.001

*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
DESIGN NASHYD       ID=1 NHYD=101 DT=5 MIN AREA=0.88 HA DWF=0 CN=85 TP=0.22
                    END=-1
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
DESIGN NASHYD       ID=2 NHYD=102 DT=5 MIN AREA=4.18 HA DWF=0 CN=65 TP=0.33
                    END=-1

ADD HYD             ID=3 NHYD=111  IDONE=1  IDTWO=2

*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
DESIGN NASHYD       ID=4 NHYD=104 DT=5 MIN AREA=4.45 HA DWF=0 CN=75 TP=0.29
                    END=-1

ADD HYD             ID=5 NHYD=112  IDONE=3  IDTWO=4

*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
DESIGN NASHYD       ID=6 NHYD=106 DT=5 MIN AREA=2.55 HA DWF=0 CN=77 TP=0.38
                    END=-1

ADD HYD             ID=7 NHYD=113  IDONE=5  IDTWO=6

*
* ------------------------------------------------
*  TEMP FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
DESIGN NASHYD       ID=8 NHYD=108 DT=5 MIN AREA=4.54 HA DWF=0 CN=77 TP=0.32
                    END=-1

ADD HYD             ID=9 NHYD=114  IDONE=7  IDTWO=8
```

```
*
* ------------------------------------------
*  RE-RUN for AES and SCS events
* ------------------------------------------
*
START               TIME=0 METOUT=0 NSTORM=1 NRUN=2
                    100AES12.STM

START               TIME=0 METOUT=0 NSTORM=1 NRUN=3
                    100SCS12.STM
*
FINISH
```

```
 OOO   TTTTT  TTTTT  H   H  Y   Y  M   M   OOO    I N T E R H Y M O
O   O    T      T    H   H   Y Y   MM MM  O   O   * * * 1989b * * *
O   O    T      T    HHHHH    Y    M M M  O   O
O   O    T      T    H   H    Y    M   M  O   O
 OOO     T      T    H   H    Y    M   M   OOO    cE-314741600002
```


Input filename: 220MPRE.TXT
Output filename: 220MPRE.OUT
Summary filename: 220MPRE.SUM

DATE: 04-27-2025 TIME: 14:11:47

COMMENTS: _____

```
*******************************************************************************
*                                                                             *
*                    SKIRA & ASSOCIATES LIMITED                               *
*                          MARCH   2020                                       *
*                   DEZEN INDUSTRIAL   SUBDIVISION                            *
*                        PRE DEVELOPMENT                                      *
*                STORMWATER MANAGEMENT PRACTICES                              *
*                                                                             *
*******************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                                 *
*  100 YEAR DESIGN STORMS                                                     *
*******************************************************************************
  ****************************
  ** SIMULATION NUMBER:   1 **
  ****************************
```

READ STORM	Filename: 100MIS4.CHI
Ptotal= 79.43 mm	Comments: CHICAGO - 100 YR STORM DISTRIBUTION - MI

TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr	TIME hrs	RAIN mm/hr
.08	4.98	1.08	21.69	2.08	13.80	3.08	6.62
.17	5.28	1.17	33.28	2.17	12.57	3.17	6.37
.25	5.62	1.25	76.62	2.25	11.55	3.25	6.13
.33	6.02	1.33	242.53	2.33	10.71	3.33	5.92
.42	6.49	1.42	98.69	2.42	9.98	3.42	5.72
.50	7.05	1.50	54.64	2.50	9.36	3.50	5.54
.58	7.72	1.58	37.73	2.58	8.82	3.58	5.37
.67	8.57	1.67	28.91	2.67	8.35	3.67	5.21
.75	9.66	1.75	23.53	2.75	7.92	3.75	5.06
.83	11.12	1.83	19.90	2.83	7.55	3.83	4.92
.92	13.17	1.92	17.30	2.92	7.21	3.92	4.78
1.00	16.30	2.00	15.34	3.00	6.90	4.00	4.66

```
*
* ------------------------------------------------
```

```
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN          |
| NASHYD    (0101) |   Area    (ha)=    .88   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .22

     Unit Hyd Qpeak  (cms)=    .153

     PEAK FLOW       (cms)=    .138 (i)
     TIME TO PEAK    (hrs)=   1.583
     RUNOFF VOLUME    (mm)=  49.403
     TOTAL RAINFALL   (mm)=  79.430
     RUNOFF COEFFICIENT   =    .622

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN          |
| NASHYD    (0102) |   Area    (ha)=   4.18   Curve Number   (CN)= 65.0
|ID= 2 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .33

     Unit Hyd Qpeak  (cms)=    .484

     PEAK FLOW       (cms)=    .268 (i)
     TIME TO PEAK    (hrs)=   1.750
     RUNOFF VOLUME    (mm)=  28.278
     TOTAL RAINFALL   (mm)=  79.430
     RUNOFF COEFFICIENT   =    .356

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0111) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
-------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 1 (0101):      .88      .14     1.58    49.40
       + ID2= 2 (0102):     4.18      .27     1.75    28.28
         ================================================
         ID = 3 (0111):     5.06      .39     1.67    31.95

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN          |
| NASHYD    (0104) |   Area    (ha)=   4.45   Curve Number   (CN)= 75.0
|ID= 4 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .29
```

```
     Unit Hyd Qpeak  (cms)=    .586

     PEAK FLOW       (cms)=    .425 (i)
     TIME TO PEAK    (hrs)=   1.667
     RUNOFF VOLUME    (mm)=  37.333
     TOTAL RAINFALL   (mm)=  79.430
     RUNOFF COEFFICIENT   =    .470

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0112) |
|   3 +  4 =  5    |         AREA    QPEAK    TPEAK     R.V.
-------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 3 (0111):     5.06      .39     1.67    31.95
       + ID2= 4 (0104):     4.45      .42     1.67    37.33
         ================================================
         ID = 5 (0112):     9.51      .82     1.67    34.47

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN           |
| NASHYD    (0106) |   Area    (ha)=   2.55   Curve Number   (CN)= 77.0
|ID= 6 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .38

     Unit Hyd Qpeak  (cms)=    .256

     PEAK FLOW       (cms)=    .217 (i)
     TIME TO PEAK    (hrs)=   1.750
     RUNOFF VOLUME    (mm)=  39.478
     TOTAL RAINFALL   (mm)=  79.430
     RUNOFF COEFFICIENT   =    .497

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0113) |
|   5 +  6 =  7    |         AREA    QPEAK    TPEAK     R.V.
-------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 5 (0112):     9.51      .82     1.67    34.47
       + ID2= 6 (0106):     2.55      .22     1.75    39.48
         ================================================
         ID = 7 (0113):    12.06     1.03     1.67    35.53

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  TEMP FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
```

```
-------------------
| DESIGN          |
| NASHYD    (0108) |   Area    (ha)=   4.54   Curve Number   (CN)= 77.0
|ID= 8 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .32

     Unit Hyd Qpeak  (cms)=    .542

     PEAK FLOW       (cms)=    .434 (i)
     TIME TO PEAK    (hrs)=   1.667
     RUNOFF VOLUME    (mm)=  39.474
     TOTAL RAINFALL   (mm)=  79.430
     RUNOFF COEFFICIENT   =    .497

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0114) |
|   7 +  8 =  9    |         AREA    QPEAK    TPEAK     R.V.
-------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 7 (0113):    12.06     1.03     1.67    35.53
       + ID2= 8 (0108):     4.54      .43     1.67    39.47
         ================================================
         ID = 9 (0114):    16.60     1.46     1.67    36.61

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------
*  RE-RUN for AES and SCS events
* ------------------------------------------
*
  ** END OF SIMULATION :   1

********************************************************************************




  ****************************
  ** SIMULATION NUMBER:   2 **
  ****************************

********************************************************************************
*                                                                              *
*                     SKIRA & ASSOCIATES LIMITED                               *
*                           MARCH   2020                                       *
*                    DEZEN INDUSTRIAL   SUBDIVISION                            *
*                         PRE DEVELOPMENT                                      *
*                  STORMWATER MANAGEMENT PRACTICES                             *
*                                                                              *
********************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                                  *
*  100 YEAR DESIGN STORMS                                                      *
********************************************************************************

-------------------
|    READ STORM    |    Filename: 100AES12.STM
| Ptotal= 88.54 mm |    Comments: 100yr/12hr
-------------------
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
```

hrs	mm/hr	hrs	mm/hr	hrs	mm/hr	hrs	mm/hr
.25	.00	3.50	15.05	6.75	6.20	10.00	.89
.50	.89	3.75	15.05	7.00	6.20	10.25	.89
.75	.89	4.00	15.05	7.25	6.20	10.50	.89
1.00	.89	4.25	15.05	7.50	3.54	10.75	.89
1.25	.89	4.50	40.71	7.75	3.54	11.00	.89
1.50	.89	4.75	40.71	8.00	3.54	11.25	.89
1.75	.89	5.00	40.71	8.25	3.54	11.50	.89
2.00	.89	5.25	40.71	8.50	1.77	11.75	.89
2.25	.89	5.50	11.51	8.75	1.77	12.00	.89
2.50	5.31	5.75	11.51	9.00	1.77	12.25	.89
2.75	5.31	6.00	11.51	9.25	1.77		
3.00	5.31	6.25	11.51	9.50	.89		
3.25	5.31	6.50	6.20	9.75	.89		

---

*
* ---
* FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ---
*

DESIGN					
NASHYD (0101)	Area	(ha)=	.88	Curve Number (CN)=	85.0
ID= 1 DT= 5.0 min	Ia	(mm)=	1.50	# of Linear Res.(N)=	3.00
	U.H. Tp(hrs)=		.22		

NOTE: RAINFALL WAS TRANSFORMED TO 5.0 MIN. TIME STEP.

Unit Hyd Qpeak (cms)= .153

PEAK FLOW (cms)= .076 (i)
TIME TO PEAK (hrs)= 5.250
RUNOFF VOLUME (mm)= 57.371
TOTAL RAINFALL (mm)= 88.540
RUNOFF COEFFICIENT = .648

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

---

*
* ---
* FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ---
*

DESIGN					
NASHYD (0102)	Area	(ha)=	4.18	Curve Number (CN)=	65.0
ID= 2 DT= 5.0 min	Ia	(mm)=	1.50	# of Linear Res.(N)=	3.00
	U.H. Tp(hrs)=		.33		

Unit Hyd Qpeak (cms)= .484

PEAK FLOW (cms)= .197 (i)
TIME TO PEAK (hrs)= 5.333
RUNOFF VOLUME (mm)= 33.840
TOTAL RAINFALL (mm)= 88.540
RUNOFF COEFFICIENT = .382

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

```
| ADD HYD   (0111) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 1 (0101):      .88      .08     5.25    57.37
       + ID2= 2 (0102):     4.18      .20     5.33    33.84
         ================================================
         ID = 3 (0111):     5.06      .27     5.25    37.93

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN          |
| NASHYD    (0104) |   Area    (ha)=   4.45   Curve Number   (CN)= 75.0
|ID= 4 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .29

     Unit Hyd Qpeak  (cms)=    .586

     PEAK FLOW       (cms)=    .283 (i)
     TIME TO PEAK    (hrs)=   5.250
     RUNOFF VOLUME    (mm)=  44.100
     TOTAL RAINFALL   (mm)=  88.540
     RUNOFF COEFFICIENT   =    .498

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0112) |
|   3 +  4 =  5    |         AREA    QPEAK    TPEAK     R.V.
------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 3 (0111):     5.06      .27     5.25    37.93
       + ID2= 4 (0104):     4.45      .28     5.25    44.10
         ================================================
         ID = 5 (0112):     9.51      .55     5.25    40.82

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN          |
| NASHYD    (0106) |   Area    (ha)=   2.55   Curve Number   (CN)= 77.0
|ID= 6 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .38

     Unit Hyd Qpeak  (cms)=    .256

     PEAK FLOW       (cms)=    .160 (i)
     TIME TO PEAK    (hrs)=   5.333
     RUNOFF VOLUME    (mm)=  46.493
     TOTAL RAINFALL   (mm)=  88.540
     RUNOFF COEFFICIENT   =    .525
```

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

ADD HYD (0113)		AREA	QPEAK	TPEAK	R.V.
5 + 6 = 7		(ha)	(cms)	(hrs)	(mm)
	ID1= 5 (0112):	9.51	.55	5.25	40.82
	+ ID2= 6 (0106):	2.55	.16	5.33	46.49
	ID = 7 (0113):	12.06	.71	5.33	42.02

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

---

*
* ----------
* TEMP FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ----------
*

DESIGN NASHYD (0108)	ID= 8 DT= 5.0 min				
		Area (ha)=	4.54	Curve Number (CN)=	77.0
		Ia (mm)=	1.50	# of Linear Res.(N)=	3.00
		U.H. Tp(hrs)=	.32		

Unit Hyd Qpeak (cms)= .542

PEAK FLOW (cms)= .298 (i)
TIME TO PEAK (hrs)= 5.333
RUNOFF VOLUME (mm)= 46.488
TOTAL RAINFALL (mm)= 88.540
RUNOFF COEFFICIENT = .525

(i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

ADD HYD (0114)		AREA	QPEAK	TPEAK	R.V.
7 + 8 = 9		(ha)	(cms)	(hrs)	(mm)
	ID1= 7 (0113):	12.06	.71	5.33	42.02
	+ ID2= 8 (0108):	4.54	.30	5.33	46.49
	ID = 9 (0114):	16.60	1.01	5.33	43.24

NOTE: PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

---

*
* ----------
* RE-RUN for AES and SCS events
* ----------
*
** END OF SIMULATION : 2

****

****

```
  ** SIMULATION NUMBER:   3 **
  ***************************

*************************************************************************
*                                                                       *
*                    SKIRA & ASSOCIATES LIMITED                         *
*                          MARCH   2020                                 *
*                   DEZEN INDUSTRIAL   SUBDIVISION                      *
*                        PRE DEVELOPMENT                                *
*                STORMWATER MANAGEMENT PRACTICES                        *
*                                                                       *
*************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                           *
*  100 YEAR DESIGN STORMS                                               *
*************************************************************************

-------------------
|    READ STORM    |    Filename: 100SCS12.STM
| Ptotal= 17.53 mm |    Comments: 100yr/12hr
-------------------
                TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                 hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr
                 .08     .00 |   .42   12.90 |   .75   12.90 |  1.08   12.90
                 .17    5.95 |   .50   16.86 |   .83   12.90 |
                 .25    3.97 |   .58   12.90 |   .92   52.58 |
                 .33    5.95 |   .67   22.82 |  1.00   37.70 |

-------------------------------------------------------------------------

*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN           |
| NASHYD    (0101) |   Area    (ha)=    .88   Curve Number   (CN)= 85.0
|ID= 1 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .22

     Unit Hyd Qpeak  (cms)=    .153

     PEAK FLOW       (cms)=    .020 (i)
     TIME TO PEAK    (hrs)=   1.083
     RUNOFF VOLUME    (mm)=   4.209
     TOTAL RAINFALL   (mm)=  17.528
     RUNOFF COEFFICIENT   =    .240

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

-------------------------------------------------------------------------

*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN           |
| NASHYD    (0102) |   Area    (ha)=   4.18   Curve Number   (CN)= 65.0
|ID= 2 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .33

     Unit Hyd Qpeak  (cms)=    .484

     PEAK FLOW       (cms)=    .028 (i)
```

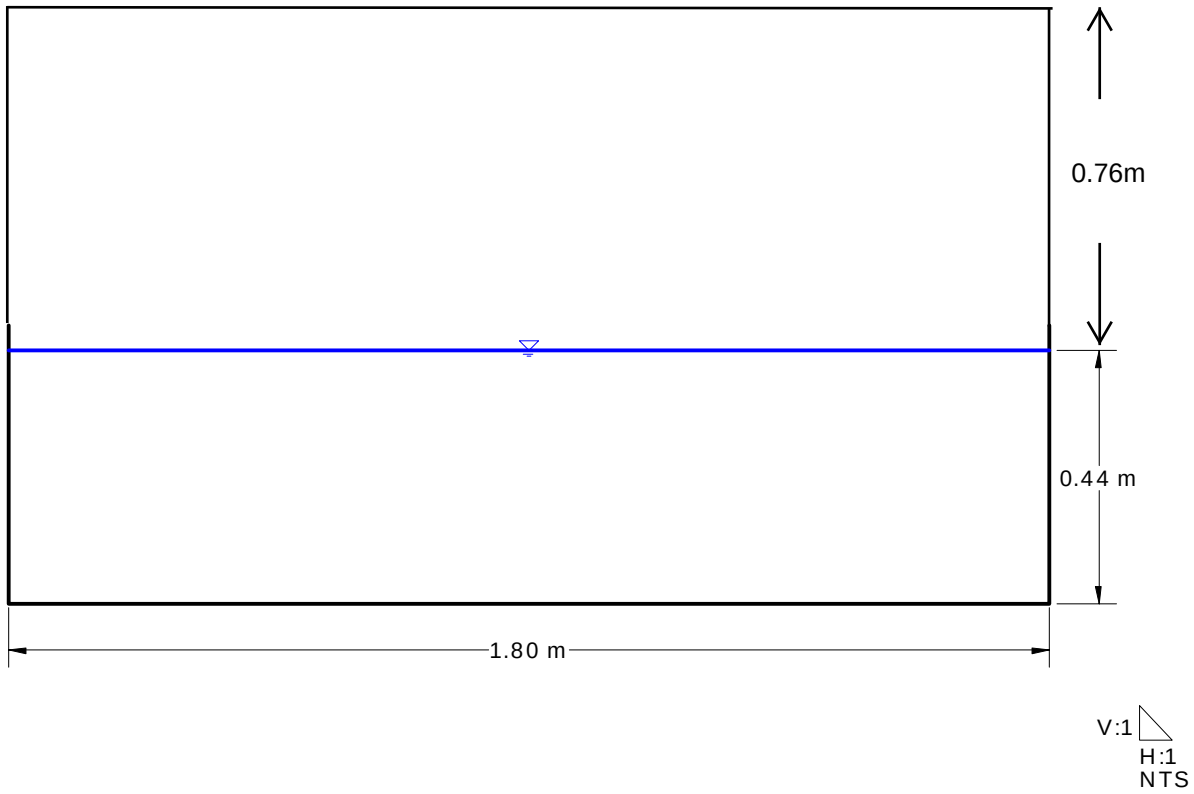
```
     TIME TO PEAK    (hrs)=   1.250
     RUNOFF VOLUME    (mm)=   1.679
     TOTAL RAINFALL   (mm)=  17.528
     RUNOFF COEFFICIENT   =    .096

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0111) |
|   1 +  2 =  3    |         AREA    QPEAK    TPEAK     R.V.
------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 1 (0101):      .88      .02     1.08     4.21
       + ID2= 2 (0102):     4.18      .03     1.25     1.68
         ================================================
         ID = 3 (0111):     5.06      .05     1.17     2.12

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN           |
| NASHYD    (0104) |   Area    (ha)=   4.45   Curve Number   (CN)= 75.0
|ID= 4 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .29

     Unit Hyd Qpeak  (cms)=    .586

     PEAK FLOW       (cms)=    .051 (i)
     TIME TO PEAK    (hrs)=   1.167
     RUNOFF VOLUME    (mm)=   2.549
     TOTAL RAINFALL   (mm)=  17.528
     RUNOFF COEFFICIENT   =    .145

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0112) |
|   3 +  4 =  5    |         AREA    QPEAK    TPEAK     R.V.
------------------          (ha)    (cms)    (hrs)     (mm)
         ID1= 3 (0111):     5.06      .05     1.17     2.12
       + ID2= 4 (0104):     4.45      .05     1.17     2.55
         ================================================
         ID = 5 (0112):     9.51      .10     1.17     2.32

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------
*  FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------
*
-------------------
| DESIGN           |
| NASHYD    (0106) |   Area    (ha)=   2.55   Curve Number   (CN)= 77.0
|ID= 6 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
```

```
------------------   U.H. Tp(hrs)=    .38

     Unit Hyd Qpeak  (cms)=    .256

     PEAK FLOW       (cms)=    .026 (i)
     TIME TO PEAK    (hrs)=   1.250
     RUNOFF VOLUME    (mm)=   2.792
     TOTAL RAINFALL   (mm)=  17.528
     RUNOFF COEFFICIENT   =    .159

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0113) |
|   5 +  6 =  7    |         AREA    QPEAK    TPEAK     R.V.
-------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 5 (0112):     9.51      .10     1.17     2.32
        + ID2= 6 (0106):     2.55      .03     1.25     2.79
          ================================================
          ID = 7 (0113):    12.06      .12     1.17     2.42

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------------
*  TEMP FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------------
*
-------------------
| DESIGN           |
| NASHYD    (0108) |   Area    (ha)=   4.54   Curve Number   (CN)= 77.0
|ID= 8 DT= 5.0 min |   Ia      (mm)=   1.50   # of Linear Res.(N)= 3.00
-------------------   U.H. Tp(hrs)=    .32

     Unit Hyd Qpeak  (cms)=    .542

     PEAK FLOW       (cms)=    .052 (i)
     TIME TO PEAK    (hrs)=   1.167
     RUNOFF VOLUME    (mm)=   2.793
     TOTAL RAINFALL   (mm)=  17.528
     RUNOFF COEFFICIENT   =    .159

     (i) PEAK FLOW DOES NOT INCLUDE BASEFLOW IF ANY.

--------------------------------------------------------------------------------

-------------------
| ADD HYD   (0114) |
|   7 +  8 =  9    |         AREA    QPEAK    TPEAK     R.V.
-------------------          (ha)    (cms)    (hrs)     (mm)
          ID1= 7 (0113):    12.06      .12     1.17     2.42
        + ID2= 8 (0108):     4.54      .05     1.17     2.79
          ================================================
          ID = 9 (0114):    16.60      .18     1.17     2.52

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* -------------------------------------------
*  RE-RUN for AES and SCS events
* -------------------------------------------
```

Cross Section
Cross Section for Rectangular Channel

Project Description	
Worksheet	Rectangular Chann
Flow Element	Rectangular Chann
Method	Manning's Formula
Solve For	Channel Depth

Section Data		
Mannings Coeffic	0.013	
Slope	000500	m/m
Depth	0.44	m
Bottom Width	1.80	m
Discharge	0.6000	m³/s



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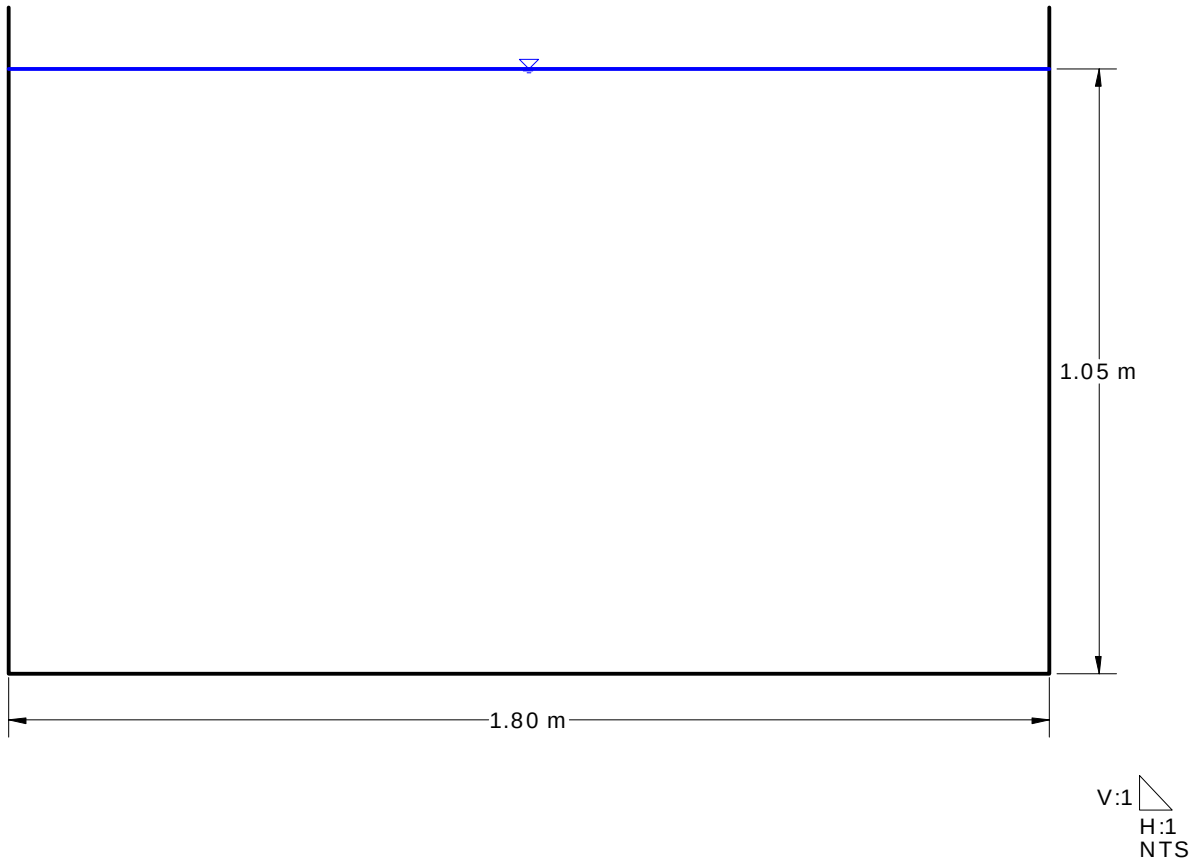

Tributary of Fletcher's Creek Fluvial Geomorphological Assessment

DeZen Lands Development

7140 Hurontario Street, Mississauga

Prepared for:
DeZen Realty
128 Queen Street South
Mississauga, Ontario L5M 1K8

Submitted:
September 12, 2025

GEO Morphix Project No. 25102

Ver.	Purpose/Change	Authored by	Approved by	Date
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Disclaimer

This report presents professional opinions and findings of a scientific and technical nature based on the knowledge and information available at the time of preparation. This document is prepared solely for the Client, and the data, interpretations, suggestions, recommendations, and opinions expressed in the report pertains only to the project being completed for the Client.

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1 Introduction

GEO Morphix Ltd. was retained to provide a geomorphological assessment for the proposed DeZen Lands Development located at 7140 Hurontario Street in the City of Mississauga, Ontario. Two watercourses are located within or adjacent to the subject lands, Fletcher's Creek and a small tributary of Fletcher's hereafter referred to as Tributary 1. The geomorphological assessment provides guidance in addressing and delineating erosion hazards and supporting erosion mitigation strategies for the stormwater management plan.

To fulfill erosion hazard delineation requirements, a toe erosion allowance was determined to support the slope stability assessment completed by Soil Engineers Ltd for Fletcher's Creek and Tributary 1. A short section of channel within Tributary 1 was identified as being unconfined as such, a meander belt width was also calculated for this reach. The meander belt width previously delineated for Fletchers Creek by Parish (2011) was also reviewed and further refined.

To support erosion mitigation, a crossing assessment and conceptual outfall design has been completed and integrated into this report. The location of the proposed outfall and crossing location were assessed in the field to support the assessment and conceptual design, respectively.

The following activities were completed as part of the geomorphological assessment in support of the development plan:

- Background review of reports and mapping for the subject lands (i.e., watershed/subwatershed studies, geology, topography, conceptual development plans, past environmental reports)
- Delineate watercourse reaches based on a desktop assessment of available data and confirmed through field reconnaissance
- Review of historical and recent aerial photographs to assess alterations in channel planform and location of the toe of slope over time
- Reach-level rapid geomorphological field assessments following standard protocols (e.g., RGA, RSAT) to evaluate instream and riparian conditions, and overall stability of the channel
- Delineate or refine limits of the meander belt width/erosion hazard on a reach basis using results of the desktop and field assessments
- A detailed geomorphic assessment including a longitudinal survey of the channel center line, 8 channel cross-section surveys, and Wolman pebble counts to determine grain size and material type for channel bed and banks
- Provide technical input and recommendations for the watercourse crossing. This includes input on watercourse crossing size and location as well as any setback limits in support of erosion mitigation
- Provide technical support for the assessment of any outlet locations to assess local erosion
- Provide support in development of a conceptual outfall design

2 Background Review

The subject lands are situated within the Fletchers Creek Subwatershed of the overall Credit River Watershed. The Fletchers Creek and associated tributary are present within or adjacent to the subject lands. Land use within the watershed is dominated by approximately 35% of natural land cover, 31% urban land cover, and 34 % agricultural and open space. The natural areas include forests, wetlands, meadows, and riparian areas (Credit Valley Conservation). The subject lands are generally comprised of agricultural lands with Tributary 1 flowing south through the subject lands and Fletchers Creek flowing in a southern direction south of the site. Fletchers Creek is also noted as occupied Redside dace habitat.

2.1 Background Reports

A detailed review of the documents below was conducted to understand relevant information associated with the watercourses for current or future assessments. The studies provide insight into past historical information, rapid assessment information and overall erosion potential information.

- DRAFT DeZen: Fletchers Creek Hazard Assessment – Parish Geomorphic (2011)

An erosion hazard assessment was completed to support the study area which consisted of background review, desktop assessment, rapid assessments, meander belt width delineation, and channel migration analysis for Fletchers Creek and its tributary. Reaches were assessed through field verified rapid assessments completed using RGA and RSAT tools and all reaches were reported to be in transition with the dominant process of widening.

A preliminary meander belt width was delineated along Reaches **FC-1, FC-2, FC-3, FCT-1**, and **FCT-2** (Tributary) using the Leopold and Wolman Method (1960). The method involves measuring the widest meander amplitudes along the reach to determine the meander belt width. Empirically modelling was also completed to estimate the meander belt widths including Williams (1986), Ward (2002), PARISH Geomorphic Ltd. (2004a) and Annable (1996). Ultimately, the meander belt widths determined by the Leopold & Wolman method was determined to be the most appropriate. An erosion analysis and channel migration rate was also completed along the main branch which studied rates at which the channel is migrating over time through historical aerial photograph review. The tributary was densely vegetated and as such, migration rates were not calculated. The 100-year erosion migration rate was determined to be 8 metres. Ultimately, the erosion hazard was determined by the meander belt width due to the inherent error in the migration rate measurements, field observations, and as a conservative approach given the meander belt width was equal to or slightly higher than the estimated 100-year erosion rate.

The results of the erosion hazard assessment are summarized as follows:

Table 1: Erosion Hazard Assessment results (Parish, 2011)

Reach Names	Preliminary Meander Belt Widths*	Erosion Migration Rate (m per 100 yrs)
FC-1	72	8
FC-2	108	8
FC-3	120	8
Tributary (FCT-1 & FCT-2)	22	n/a

**Including a FOS of 20%*

- Detailed Geomorphic Assessment – Fletchers Creek – Geomorphic Solutions (2012)

A detailed geomorphic survey was completed along a portion of Fletchers Creek downstream of the tributary in support of Redside Dace habitat delineation. The detailed geomorphic survey extended 278 m downstream of the tributary confluence with an average bankfull width and depth of 8.5 m and 0.7 m respectively. This assessment supported the assessment that the tributary is not contributing Redside Dace habitat as the average bankfull width downstream of the tributary confluence are more 7.5 m wide as discussed in the EIS (GEI, 2025).

- Environmental Impact Study (EIS) – 7140 Hurontario Street, Mississauga – GEI Consultants Canada Ltd. (February, 2025)

An EIS was completed and submitted as part of the resubmission for the draft plan of subdivision and zoning by-law amendment. The study was comprised of existing condition characterization of the natural environmental features, constraint delineation, identification of potential impacts of the development, and mitigation recommendations with the input of multiple disciplines and consultants.

With respect to fluvial geomorphology, the erosion hazard was one component of the constraint mapping which was based on the meander belt width previously delineated by Parish (2011) and the toe erosion

allowance recommended by GEO Morphix Ltd as discussed in **Section 4.2**. The classification of Redside Dace habitat was also discussed within EIS as previously noted.

- Functional Servicing Report (FSR) – 7140 Hurontario Street, Mississauga – Skira and Consultants Ltd. (February, 2025)

An FSR was completed and submitted as part of the resubmission for the draft plan of subdivision and zoning by-law amendment. The study was comprised of existing site condition characterization, grading plans, stormwater management plan, servicing requirements, and erosion and sediment controls.

The site will be developed in two Phases with the lands east of the tributary developed in Phase 1 and the lands west of the tributary will be developed in Phase 2. The Tributary to Fletchers Creek will continue to receive flows from approximately 12 ha of drainage area north of the subject lands in the post development scenarios. In Phase 1, flows will be conveyed to the existing storm sewer on Vicksburg Drive, which discharges to SWM Pond 4402B. In Phase 2 flows will generally be piped and conveyed to an outfall to the main branch of Fletchers Creek. A treatment train approach will be utilized to retain the first 5 mm of rainfall and for quality control.

- Geotechnical Investigation for Slope Stability Study – Soil Engineers Ltd. (2008)

  Supplementary Slope Stability Study Letter Report – Soil Engineers Ltd. (2016)

  Supplementary Slope Stability Study Letter Report – Soil Engineers Ltd. (2020)

A slope stability assessment was completed in 2008 and subsequently updated in 2016 and 2020 after receiving comments from agencies. The initial assessment provided characterization of the subsurface conditions and groundwater conditions, and slope stability analysis. It was determined that the site was generally comprised of silty clay till soils with a localized layer of very dense sandy silt till in the areas of the Fletchers Creek Tributary (BH1 and BH2). All boreholes remained dry upon completion of the investigation and the groundwater regime was inferred to lie in the grey saturated soils. A long-term stable slope was delineated which incorporated a 5 m development setback and an 8 m toe erosion setback.

In 2016, the cross sections were updated to reflect an updated topographic survey. In 2020, the tributary to Fletchers Creek was also added to the slope study. Additionally, the long-term stable slope was updated to incorporate the recommended toe erosion setbacks and meander belt widths determined by GEO Morphix Ltd. as discussed in **Section 4**.

2.2 Surficial Geology

Channel morphology and planform are largely governed by the flow regime and the availability and type of sediments (i.e., surficial geology) within the stream corridor. Physiography, riparian vegetation, and land use also physically influence the channel. These factors provide insight into existing conditions and the potential future changes as they relate to a proposed activity.

Local surficial geology along the Tributary to Fletcher's Creek is fine-textured till composed of silt and clay derived from glaciolacustrine deposits (OGS, 2010). The study area is located within the Peel Plain physiographic region, a bevelled till plain which is characterized by gently undulating to rolling topography with layers of thick till deposits on bedrock (Chapman and Putnam, 1984). Understanding the surficial geology of the study area is important for determining the toe erosion allowance and assessing the erosion hazard, as stability of the channel banks and valley slope is dependent on soil composition and structure (MNRF, 2001).

2.3 Historical Assessment

A series of historical aerial photographs were reviewed to determine changes to the channel and surrounding land use/cover. This information, in part, provides an understanding of the historical factors that have contributed to current channel morphodynamics. For this exercise, the 1954, 1977 and 1989 photographs were retrieved from the National Air Photo Library, and the 2002, 2018, 2023 images were

retrieved from Google Earth Pro. Cropped aerial photographs, showing Fletcher's Creek, Tributary 1 and surrounding area are provided in **Appendix B**.

In 1954 land use within the vicinity of Fletcher's Creek and Tributary 1 was almost exclusively agricultural. A riparian buffer for the tributary was absent, while adjacent to Fletcher's Creek some patches of mature trees particularly along the west side of the channel within the valley were noted. Tributary 1 was observed to originate at a small agricultural pond located on the north side of an east-west road which transects the subject property. The tributary appears to have been straightened until reaching the valley corridor, at which point the channel follows the south westerly valley trend before forming a confluence with Fletcher's Creek. The planform of Fletcher's Creek nearby to the subject site is best described as irregular meanders within a confined valley. Nearby to the confluence of Tributary 1 with Fletcher's Creek, on the opposite bank, a cut-off channel can be observed, possible evidence of planform adjustments.

In 1977 land use nearby to the subject site has remained predominantly agricultural, with the exception of the lands south of Fletcher's Creek opposite to Tributary 1 which is now occupied by a golf course. The construction of the golf course included the removal of the cut off channel noted in the 1955 aerial photograph. The only significant change related to Tributary 1 is the removal of the east-west road noted to transect previously noted to cross the tributary.

The 1989 aerial photo indicates minimal changes to land use nearby to the subject site, although it is known that lands upstream in the Fletcher's Creek catchment have become significantly urbanized by this time. For Tributary 1, the watercourse appears to be recovering from previous channelization to have a small degree of sinuosity upstream of the confined segment. Within the confined segment, the channel has maintained its irregular meanders and more woody vegetation has become established. For the mainstem of Fletcher's Creek, a meander previously located upstream of the eastward valley trend turn is no longer present. As well, a historic channel is apparent north of the first meander bend located where Fletcher's Creek conveys flow eastward towards the tributary.

By 2002 land use in the vicinity of the DeZen Property has shifted to predominantly residential and commercial, although the property itself and the adjacent lands have remained agricultural. both channels, the presence of woody riparian vegetation has increased. The only notable change to Tributary 1 is that the pond located at the upstream extent no longer holds standing water but rather exists as a wetland feature.

In 2018 the trend towards residential and commercial development has persisted, with construction of commercial properties on the west side of Hurontario and the west side of Fletcher's Creek. Riparian conditions along both channels have continued to establish, with a dense canopy now apparent within the confined section of the tributary, and a significant quantity of mature trees within the corridor of the mainstem of Fletcher's Creek. For both channels, no planform changes were apparent.

Between 2018 and 2023, the subject lands east of Tributary 1 undergo some industrial development as well. Changes along Tributary 1 and Fletcher's Creek were not observed.

3 Watercourse Characteristics

3.1 Reach Delineation

Reaches are homogeneous segments of channels used in geomorphological investigations. Reaches are divided as such because they are expected to have similar inputs and outputs in terms of sediments and discharge. They are also expected to react similarly throughout to flow events and other stressors. They are studied semi-independently as each is expected to function in a manner that is at least slightly different from adjoining reaches. This allows for a meaningful characterization of a watercourse as the aggregate of reaches, or an understanding of a particular reach, for example, as it relates to a proposed activity.

Reaches are delineated based on changes in the following:

- Channel planform

- Channel gradient
- Physiography
- Land cover (land use or vegetation)
- Flow, due to tributary inputs
- Soil type and surficial geology
- Certain types of anthropogenic channel modifications

Reach delineation follows scientifically defensible methodology proposed by Montgomery and Buffington (1997), Richards et al. (1997), and the Toronto and Region Conservation Authority (2004) as well as others.

Reaches on and adjacent to the subject lands were delineated by a desktop exercise and were subsequently verified in the field. Reach **FC-2** was delineated along the main branch of Fletchers Creek and Reaches **FCT-1, FCT-2,** and **FCT-3** were delineated along the Fletcher's Creek tributary (Tributary 1) based on changes in gradient, land use, land cover, geology, and various flow or tributary inputs. Our reach delineation is graphically presented in **Appendix A.**

The upstream extent of Reach **FCT-1** was located at the northern edge of the forest patch within the subject property between two agricultural fields. The reach conveyed flow south for approximately 200 m towards Fletcher's Creek. Reach **FCT-1** was defined by deciduous trees in the riparian zone and valley confinement on both sides of the creek. Upstream of the forest patch, Reach **FCT-2** was a short, poorly defined channel which conveyed flows southwards through the vegetated corridor separating the two agricultural fields, upstream of Reach **FCT-1**. The channel was partially confined and generally had herbaceous vegetation and grasses occupying the riparian buffer. Reach **FCT1-3** was an undefined feature which conveyed flows from the old farm pond location to the top of **FCT-2**.

3.2 General Reach Observations

Field investigations were completed on November 1, 2018, and updated on July 18, 2025, and included the following reach-by-reach observations:

- Descriptions of riparian conditions
- Estimates of bankfull channel dimensions
- Determination of bed and bank material composition and structure
- Observations of erosion, scour, or deposition
- Collection of photographs to document the watercourses, riparian areas and/or valley, surrounding land use, and channel disturbances such as crossing structures

These observations and measurements are summarized below. The descriptions are supplemented and supported with representative photographs, which are included in **Appendix C**. Field sheets, including reach summaries and rapid assessments, are provided in **Appendix D**.

Reach **FC-2** is a meandering channel within a well-defined confined valley system. The channel has a wide continuous riparian buffer zone consisting of predominately deciduous trees. The average bankfull width and depth were 8.04 m and 1.01 m, respectively. Erosion was observed throughout the reach with some areas of valley wall contact and with undercuts measured up to 0.86 m. Bed materials ranged from gravel to boulders in riffles, and clay, silt to cobbles in pools. A range of bank materials were observed due to the presence of a suspended armour layer. A high density of woody debris was also observed.

Reach **FCT-1** is a mixed-load sinuous channel with a steep gradient that occupies a confined valley. The channel was well defined but appeared to be slightly more incised in the upstream portion. The channel has a continuous riparian buffer zone that contained predominately deciduous trees with some encroached by vegetation. The average bankfull width and depth in Reach **FCT-1** were 1.21 m and 0.30 m. Erosion was noted along this reach with undercuts measured being up to 0.45 m. Bed and bank materials fairly uniform ranging from clay/silt to cobbles in riffles and in pools.

Reach **FCT-2** was a partially confined, sinuous channel with a continuous riparian buffer similar to the downstream extent. It was noted that the channel definition was observed to decrease further upstream.

The channel had no riffle-pool sequence, a moderate gradient and bed and bank material which was homogeneously composed of a range of materials from clay to cobbles. The channel was moderately encroached by riparian vegetation, and lots of woody debris was observed. Average bankfull width and depth for the reach were 1.06 m, and 0.19 m, respectively. Bank angles ranged from 30-90 degrees, and shallow undercuts were observed up to 0.14 m deep.

Reach **FCT-3** was a poorly defined feature which was predominately dry throughout. The majority of the channel was heavily encroached with herbaceous vegetation and grasses. Phragmites and cattails were observed in the upstream portion of the feature as well. The feature had no riffle-pool sequence, a low gradient and bed and bank material which was homogeneously composed of a clay-silt, sand, gravel mixture. Generally, the feature was poorly defined however bankfull measurements were collected in the portions that were defined. Average bankfull width and depth for the defined portion were 0.79 m, and 0.12 m, respectively.

3.3 Reconnaissance-level Assessments

Channel stability and susceptibility to erosion were objectively assessed through the application of the Ontario Ministry of the Environment's (2003) Rapid Geomorphic Assessment (RGA). The RGA evaluates degradation, aggradation, widening, and planimetric form adjustment at the reach scale. The end result of the RGA is to produce a score, or stability index, which evaluates the degree to which a stream has departed from its equilibrium condition. A stream with a score of less than 0.20 is in regime, indicating minimal changes to its shape or processes over time. A score of 0.21 to 0.40 indicates that a stream is in transition or stressed and is experiencing major changes to process and form outside the natural range of variability. A score of greater than 0.41 indicates that a stream is in extreme adjustment, exhibiting a new stream type, or in the process of adjusting to a new equilibrium (MOE, 2003; VANR, 2007).

The Rapid Stream Assessment Technique (RSAT) was also employed to provide a broader view of the system and consider the ecological functioning of the watercourse (Galli, 1996). Observations were made of channel stability, channel scouring or sediment deposition, instream and riparian habitats, and water quality. The RSAT score ranks the channel as maintaining a poor (<13), fair (13-24), good (25-34), or excellent (35-42) degree of stream health.

The reaches were also classified according to a modified Downs (1995) Channel Evolution Model and the River Styles Framework (Brierley and Fryirs, 2005). The Downs' Model describes successional stages of a channel as a result of a perturbation, namely hydromodification. Understanding the current stage of the system is beneficial as this allows one to predict how the channel will continue to evolve or respond to an alteration to the system. The River Styles Framework (Brierley and Fryirs, 2005) provides a geomorphological approach to examining river character, behaviour, condition, and recovery potential.

Reach **FC-2** was identified as primarily widening according to the RGA, as evidenced by the occurrence of fallen and leaning trees, large organic debris, exposed tree roots, and basal scour throughout more than 50% of the reach. Overall, the reach was assigned a stability index of 0.34 and classified as "in transition". The RSAT resulted in a *Good* ranking with a score of 27, with the limiting factor being channel stability, as evidenced by the frequently observed bank erosion also noted by the RGA. Using Downs' Model of Channel Evolution, the channel was determined to be U – undercutting, as evidenced by erosion and undercutting along the banks.

Reach **FCT-1** was identified as primarily widening with some degradation according to the RGA, as evidenced by the fallen and leaning trees, exposed tree roots, and suspended armour layer. Overall, the reach was assigned a stability index of 0.37 and classified as "in transition". The RSAT resulted in a *good* ranking with a score of 27, with the limiting factor being physical instream habitat and channel stability, as evidenced by the lack of diverse geomorphological units and bank erosion also noted by the RGA. Using Downs' Model of Channel Evolution, the channel was determined to be e - Enlarging, as evidenced by erosion along both banks and a scoured bed.

Reach **FCT-2** was identified by the RGA as being "In transition", with the score of 0.21, with the primary process being degradation and widening. This was evidenced by the suspended armour layer and leaning and fallen trees, and exposed roots. The RSAT resulted in a *fair* ranking with a score of 21, with the

limiting factor being physical instream habitat, as evidenced by the lack of diverse geomorphological units. Using Downs' Model of Channel Evolution, the channel was determined to be e - Enlarging, as evidenced by erosion along both banks.

Reach **FCT-3** was generally described as a poorly defined swale that vegetation controlled throughout the reach. As such, the RGA and RSAT tools are not applicable. According to Downs' model of Channel Evolution, the reach was classified as stable given there were almost no observations of ongoing geomorphic change.

3.4 Detailed Geomorphological Assessment

A detailed geomorphic assessment was completed within the downstream section of Reach **FCT-1**. The survey was completed on November 1, 2018. Activities completed for the detailed assessment included the following:

- Longitudinal profile of the channel bed to determine slope
- Eight representative cross-sectional surveys of the watercourse to determine average channel dimensions
- Detailed instream measurements at each cross-section including bankfull channel geometry, riparian conditions, bank materials, bank height/angle, and bank root density
- Bed material sampling at each cross-section
- Monumented geo-referenced photographs taken at each cross-section

The results of the detailed assessments are provided in **Table 2**, and a summary is included in **Appendix E**.

Table 2: Detailed geomorphic assessment results

Channel parameter	
Measured	
Average bankfull channel width (m)	2.25
Average bankfull channel depth (m)	0.50
Channel bed gradient (%)	2.55
D_{50} (mm)	3.2
D_{84} (mm)	15.0
Manning's n roughness coefficient	0.040
Computed	
Bankfull discharge (m^3/s)	3.29
Average bankfull velocity (m/s)	2.89
Unit stream power at bankfull (W/m^2)	474.20
Tractive force at bankfull (N/m^2)	164.37

4 Erosion Hazard Assessment

Most watercourses in southern Ontario have a natural tendency to develop and maintain a meandering planform, provided there are no spatial constraints. A meander belt width or erosion hazard assessment estimates the lateral extent that a meandering channel has historically occupied and will likely occupy in the future. This assessment is therefore useful for determining the potential hazard to proposed activities in the vicinity of a watercourse.

When defining the erosion hazard for a watercourse, Ministry of Natural Resources (MNR, 2002) guidelines treat unconfined and confined systems differently. Unconfined systems are those with poorly

defined valleys or slopes well outside where the channel could realistically migrate. Unconfined systems are generally found within glaciated plains with flat or gently rolling topography. Confined systems are those where the watercourse is contained within a defined valley, where valley wall contact is possible.

In unconfined systems, at minimum, a meander belt width can be applied based on 20 times the bankfull channel width. Alternatively, the limit of the erosion hazard and migration potential can be delineated based on the meander amplitude through a detailed geomorphological study. Meander amplitude is defined by Leopold et al. (1964) as the lateral distance between tangential lines drawn to the center channel of two successive meander bends. This differs from meander belt, which is measured for a reach between lines drawn tangentially to the outside bends of the laterally extreme meander bends (TRCA, 2004). Both the meander belt width and amplitude quantify the lateral extent of a river's occupation on the floodplain (TRCA, 2004).

In confined systems, the MNR outlines an approach for establishing the erosion hazard where valley walls confine watercourses. The approach defines a toe erosion allowance or setback where the channel is within 15 m of the toe of slope. There are several ways to define the toe erosion allowance or setback: using an average annual recession rate; applying a generic and minimum 15 m toe erosion allowance in areas where the channel is within 15 m of the slope's toe; or using soil information and field observations of geomorphic processes (MNRF, 2002) for areas where average annual recession rates cannot be determined.

Based on field reconnaissance and desktop information, it was determined that Reach **FCT-2** was partially confined, requiring a meander belt width on the eastern bank. The main branch of Fletchers Creek (Reach **FC-2**) and Tributary 1 (Reach **FCT-1, FCT-2**) flow within a confined or partially valley systems requiring a toe erosion allowance to address the erosion hazard. Despite being a confined valley system, a meander belt width was also delineated for Reach **FC-2** to support Redside Dace habitat delineation.

4.1 Meander Belt Width Delineation

A review of recent and historical aerial imagery was completed but due to the low resolution and a densely vegetated riparian corridor, the watercourse in Reach **FCT-2** was not traceable for historical overlay analysis. As such, a suite of empirical equations was therefore used to delineate existing condition meander belt widths.

Meander belt widths were calculated using several empirical models for comparison purposes. The bankfull channel dimensions observed during field reconnaissance were used to inform both the Williams (1986) and Ward (2002) models outlined below.

The empirical relations from Williams (1986) were modified to include channel width, and applied using the bankfull channel dimensions such that:

$$B_w = 18A^{0.65} + W_b \quad \text{[Eq. 1]}$$

$$B_w = 4.3{W_b}^{1.12} + W_b \quad \text{[Eq. 2]}$$

where *Bw* is meander belt width (m), *A* is bankfull cross-sectional area (m^2), and W_b is bankfull channel width (m). An additional 20% buffer, or factor of safety, was applied to the computed belt width values. This addresses issues of under prediction.

The Ward et al. (2002) bankfull width model was also used to determine a meander belt width (ft), *Bw*:

$$B_w = 6{W_b}^{1.12} \quad \text{[Eq. 3]}$$

The resulting value was then converted to the metric system (m). An additional 20% buffer, or factor of safety, was applied to the computed belt width values.

Empirical modelling results are summarized in **Table 3,** below. The extents of all meander belt widths are illustrated in **Appendix A**. For **Reach FCT-2** the calculated meander belt widths range from 5 m to 10 m. The meander belt width of **10 m** was recommended for **Reach FCT-2** based on values determined

using the William Area model which includes a 20% factor of safety (FOS). The value is considered conservative given the recommended meander belt width is slightly larger than the modelled meander belt widths. Additionally, there is limited erosion potential along this reach, particularly given the feature has predominantly clay bed and banks mixed with coarse material due to a suspended armour layer present.

GEO Morphix Ltd. also reviewed and refined the meander belt width previously delineated by Parish (2011) to support the Redside Dace habitat delineation. Based on their assessment a meander belt width of **108 m** was delineated along Reach **FC-2** which was adjacent to the development. The meander belt width was defined by 90 m and an erosion setback of 9 m on each side. This meander belt width was delineated based on Leopold et al. (1964) method which measure the largest meander amplitudes. GEO Morphix Ltd. updated the meander belt width to be truncated along the delineated toe of valley slope in any areas where the reach is confined by adjacent valley slopes (i.e. the meander belt width crosses the toe of slope). This is shown in **Appendix A**, in these locations, the channel cannot realistically migrate further as it is restricted by the valley slope.

As such, the recommended meander belt width for both reaches is considered to appropriately address the potential erosion hazard and Redside Dace habitat delineation.

Table 3: Summary of modelled meander belt widths for watercourse reaches for existing conditions

Reach	Modelled Meander Belt Widths			Recommended Meander Belt Width (m)
	Modified Williams (1986) Area*	Modified Williams (1986) Width*	Ward Width (2002)*	
FCT-2	9	5	9	10

** Includes 20% factor of safety*

4.2 Toe Erosion Allowance

When defining an erosion hazard for a confined or partially confined valley system, a toe erosion allowance is provided where the channel is within 15 m of the toe of slope. Field observations indicated that **Reach FC-2** and **Reach FCT-1** was confined and **Reach FCT-2** was partially confined.

Generally, the watercourse occupied a confined valley, and the valley walls were often located further than 15 m from the channel in **Reach FC-2**. Average annual recession rates are determined through meander migration analysis using historic and recent aerial photographs. An annual recession rate of approximately 8 m per 100 years was previously determined by Parish (2011) however, due to inherent error in the migration rate measurements and updated field observations we have recommended a toe erosion allowance of 5 m. Additionally, some of the aerial photographs were densely vegetated and banks were poorly visible.

Reaches FCT-1 and **FCT-2** observed on-site were densely vegetated and poorly visible through aerial photograph interpretation. Given the poor aerial coverage and limited channel definition observed in the historical and recent aerials, meander migration analysis was not possible to determine an average annual recession rate.

As such, we have developed recommendations below for an appropriate toe erosion allowance based on a combination of reach-level observations of existing geomorphic conditions and guidance outlined by MNRF in the technical guide for defining riverine erosion hazards (MNR, 2002). The recommended toe erosion allowances for each confined reach are provided in **Table 4.**

Table 4: Summary of recommended toe erosion allowances for confined valley reaches

Reach	Range of Acceptable Toe Erosion Allowances (m)*	Final Recommended Toe Erosion Allowances (m)**
FC-2	5 – 8	5
FCT-1	5 – 8	5
FCT-2	5 – 8	5

** Range based on MNR, 2002 guidelines*
*** Final toe erosion allowance based on field observations and best judgement*

The valley slopes were investigated by Soil Engineers Ltd. (2016), which found the material to be predominantly hard silty clay till and very dense sandy silt till. In accordance with the MNRF's Erosion Hazard Technical Guidelines, a minimum toe erosion allowance of 5 m is required for the watercourse based on both the valley material and the presence of active erosion within the watercourse. The toe erosion allowance informed the long-term stable slope and is depicted in the Supplementary Slope Stability Study Letter Report – Soil Engineers Ltd. (2020).

Bank materials were consistent in all three of the confined reaches. It was observed to consist primarily of clay and silt; however, a suspended amour layer was present as well which resulted in coarse materials in the banks as well. All three reaches were well vegetated and had active erosion present. **Reach FCT-2** was classified as a partially confined small channel based on field observations, with a valley slope only present on the west side of the Fletcher's Creek Tributary. The upstream portion of the reach had limited definition. **Reach FCT-1** was a small channel that was confined and well defined however, the downstream portion of the channel was observed to be smaller than the upstream portion. **Reach FC-2** was a confined channel along the main branch of Fletchers Creek. Active erosion was observed; however, erosion was not exacerbated beyond how a watercourse of its size normally functions over time. Additionally, the majority of **Reach FC-2** was not within 15 metres of the toe of slope and as such, the toe erosion allowance was limited to the downstream portion of this reach.

5 Corridor Crossing Recommendations

One crossing over Tributary 1 (**Reach FCT-3)** associated with the extension of the proposed Vicksburgh Drive. The proposed crossing will require an opening to accommodate the current feature. To accommodate potential channel adjustments, the future crossing should generally span at least three times the bankfull channel width however, this is only one consideration when assessing future channel crossing sizes. Other requirements including flood conveyance or structural requirements may require a different opening and size requirement. Additionally, it should be noted that this feature is poorly defined, vegetation controlled and lacks erosion. As such, the minimum crossing widths provided below should be considered in conjunction with other requirements.

Bankfull channel widths were collected in the field while completing the rapid assessments. Bankfull widths and proposed crossing dimensions are provided in **Table 5.**

Table 5: Bankfull width and proposed crossing widths

	Average Bankfull Width (m)	*Minimum Recommended Crossing Width (m)
FCT-3 (Vicksburgh Street Extension)	0.79	2.37

A mix of river stone and granular 'B' is proposed throughout the crossing structures to provide for a stable bed and a level of sorting. A layer of native material is also proposed to cover the bed substrate. Hydraulic sizing of all materials should be completed as a part of detailed design activities to confirm that materials are stable under a range of conditions.

6 Outfall Erosion Assessment and Design

6.1 Outfall Erosion

According to the most recent draft plan, drainage from 7.75 ha located west of Tributary 1 is proposed to be discharged into Fletcher's Creek southwest of the DeZen Lands. At this location, the Ontario Flow Assessment Tool (OFAT) measures the drainage area of Fletcher's creek as approximately 3,328 ha (33.28 km^2). Given the minor drainage contributions from the site relative to the Fletcher's Creek watershed, no exacerbated rates of erosion are expected within the watercourse as a consequence of inputs from the development.

In order to address potential local erosion resulting from site discharge, survey mapping, historical imagery was reviewed and field verified to determine a suitable location for an outfall. This review identified a historic channel located southwest of the DeZen lands which currently exists as a shallow depression. This location is considered ideal given the proximity to Fletcher's Creek, a lack of mature wooded vegetation which could be disturbed during construction and its generally stable existing condition. Erosion was not observed along the historical channel or along the valley wall in the outfall location.

6.2 Stormwater Outfall Treatment Design

A stone core naturalized energy dissipater (SCONED) with a level spreader is proposed for the SWM outlet adjacent to Fletcher's Creek. The SCONED will receive flows from the outfall, dissipating energy and providing opportunities for retention, filtration, and infiltration before water disperses through the level spreader to the historic channel. In addition to mitigating any erosion between the outlet and the receiving feature, the treatment train will provide habitat and water quality benefits. The proposed design will act as a polishing feature to further improve water quality by capturing fine sediments at the outlet. Benefits of the SCONED and level spreader system include organic inputs, temperature regulation through shade from overhanging vegetation, polishing, energy dissipation, and dispersion of flows. Additionally, the feature would provide enhanced opportunities for stormwater infiltration, evapotranspiration, and detention.

The primary objectives of the design, therefore, are to:

- Mitigate any erosion between the outlet and receiving features by dissipating energy
- Enhance the function of the SWM by providing additional water polishing
- Provide opportunities for stormwater infiltration, evapotranspiration, and detention before water reaches the receiving features
- Provide additional habitat by installing woody and herbaceous plantings around the features

The proposed SCONED should be constructed as an over-excavated depression, lined with a mix of soil and granular materials to provide depressional and subsurface storage (within the interstitial space of the sediment and soil). A stone core will be installed in the SCONED consisting of hydraulically sized rounded stone to provide additional subsurface stability at the headwall. A layer of topsoil will cover the base layer and will be seeded with the proposed wet meadow seed mix.

The level spreader proposed in conjunction with the SCONED will consist of wattles (i.e., live cuttings) and a silt sock that will be live-staked and overlain with 100% biodegradable erosion control matting. The wattles and silt sock will provide additional protection by reducing velocities and spreading flow over a wider area, promoting sheet flow to minimize any potential erosion between the SCONED and Fletcher's Creek. Wattles can withstand velocities up to 2.5 m/s, which will likely be overly sufficient to

provide stability between the SCONED and receiving feature, as the SCONED will dissipate flows from the outlets before flows pass over the level spreader (Fischenich, 2001).

An aggressive landscape restoration plan is proposed around the SCONED to provide shading over the feature. Live staking around the periphery will provide thermal mitigation through shade, create additional stability in the feature and wattles, and will also provide a source of coarse organic matter. The incorporation of a native seed mix within the wetland will promote polishing of flows once the vegetation has established.

6.2.1 Hydraulic Substrate Sizing

The proposed SWM outlet treatment should provide long-term stability and be suitably robust to withstand the anticipated flow condition. As such, the anticipated 10-year flow of 0.323 m^3/s and associated velocity of 2.71 m/s as provided by SKIRA & ASSOCIATES LTD. (2025) was used to inform the design. The stone core is expected to be stable under the predicted flow conditions in the SCONED. A layer of topsoil will be installed over the stone core to improve vegetation establishment. A substrate mix of 60% 300 mm – 400 mm diameter riverstone with 20% granular 'b' and 20% native material is proposed for the stone core at the base of the outfall. Granular 'B' consists of a mix of stone where approximately 20% – 50% of the stone is greater than 0.005 m in diameter, but nothing larger than 0.15 m in diameter.

The stone core was hydraulically sized to limit entrainment. A range of techniques was utilized to determine the appropriate stone sizing, as summarized in the National Engineering Handbook (NRCS, 2007). These techniques are provided in **Table 6**. The anticipated peak flow velocity of 2.71 m/s, provided by SKIRA & ASSOCIATES LTD (2025) was used to determine the appropriate stone size for the material. The stone size includes a factor of safety to provide additional stability. The larger stone size offers stability while allowing storage and infiltration at lower flows.

Table 6: Substrate sizes for the stone core wetland stone lining, based on a range of techniques

Model	Formula	Velocity (m/s)	Stone Size* (mm)
SCONED Core Substrate			
Isbash Method (Isbash, 1936)	$D_{50} = \left(\frac{V_c}{C * \left(2 * g * \frac{\gamma_S - \gamma_w}{\gamma_w}\right)^{0.5}}\right)^2$	2.71	368
USBR Method (Peterka, 1958)	$D_{50} = 0.0122 * V^{2.06}$	2.71	402
Maynord's Method (Maynord, 1988)	$D_{50} = C_s * C_v * C_T * d * \left[\left(\frac{\gamma_w}{\gamma_S - \gamma_w}\right)^{0.5} * \frac{V}{\sqrt{K_1 * g * d}}\right]^{2.5}$	2.71	300

*Includes 20% factor of safety

The Isbash method (Isbash, 1936) was developed for the construction of dams by placing rock into moving water. This model predicts the median stone size (D_{50}; ft) under the given flow conditions, given by:

$$D_{50} = \left(\frac{V_c}{C*\left(2*g*\frac{\gamma_S-\gamma_w}{\gamma_w}\right)^{0.5}}\right)^2 \quad \text{[Eq.1]}$$

Where:

V_c = critical velocity (ft/s)

C = Isbash constant (dimensionless)
g = gravity (ft/s)
γ_s = stone density (lb/ft^3)
γ_w = water density (lb/ft^3)

The USBR Method (Peterka, 1958) was developed for sizing riprap below a stilling basin. This model predicts the median stone size (D_{50}; ft) under the given flow conditions, given by:

$$D_{50} = 0.0122 * V^{2.06} \quad \text{[Eq.2]}$$

Where:

V = average channel velocity (ft/s)

Maynord's Method (Maynord, 1988) was developed for sizing riprap in open channel flows. This model predicts the largest stone size (D_{100}; ft) under the given flow conditions, given by:

$$D_{100} = C_s * C_v * C_T * d * [(\frac{\gamma_w}{\gamma_s - \gamma_w})^{0.5} * \frac{V}{\sqrt{K_1 * g * d}}]^{2.5} \quad \text{[Eq.3]}$$

Where:

d = water depth (ft)
C_s = stability coefficient
C_v = velocity distribution coefficient
C_T = thickness coefficient
γ_s = stone density (lb/ft^3)
γ_w = water density (lb/ft^3)
V = velocity (ft/s)
g = gravity (ft/s)
K_1 = side slope correction, calculated by:

$$K_1 = \sqrt{1 - \frac{sin^2\theta}{sin^2\emptyset}} \quad \text{[Eq.4]}$$

Where:

θ= angle of rock from the horizontal
∅ = angle of repose (typically 40°)

The values used for each variable in the Isbash method, USBR method, and Maynord's method are provided in **Table 7.**

Table 7: Variables and values associated with the stone core wetland stone sizing

Variable	HW 1
Isbash Method	
Critical velocity (V_c) (ft/s)	8.89
Isbash constant (C) (unitless)	0.86
Gravity (g) (ft/s^2)	32.2
Stone density (γ_s) (lb/ft^3)	165.43
Water density (γ_w) (lb/ft^3)	62.43

Variable	HW 1
USBR Method	
Velocity (V) (ft/s)	8.89
Maynord Method	
Water depth (d) (ft)	1.64
Stability coefficient (C_s) (unitless)	0.3
Velocity distribution coefficient (C_v) (unitless)	1
Thickness coefficient (C_T) (unitless)	1.5
Stone density (γ_s) (lb/ft^3)	165.43
Water density (γ_w) (lb/ft^3)	62.43
Velocity (V) (ft/s)	8.89
Gravity (g) (ft/s^2)	32.2
Side slope correction (K_1) (unitless)	1
Θ (°)	20
ø (°)	40

Newly constructed features can be vulnerable to erosion. This is particularly true before vegetation has been established. While low-flow events should not intensify erosion, the concern for erosion occurs when high flows or precipitation events occur during construction or prior to vegetation establishment. A 100% biodegradable erosion control blanket, native seed, and live stakes are to be installed along the level spreader and wetland perimeter for immediate erosion protection. Over time, the blanket will biodegrade, while the live stakes and native seed species will establish to provide long-term soil stability.

6.2.2 Construction Timing

Based on resident fish species and their respective life cycles, in-stream work will be regulated by the fisheries warmwater timing window (July 15th to March 15th), unless otherwise directed by the Ministry of Natural Resources (MNR).

Vegetation removals associated with clearing, site access and staging should occur outside the key breeding bird period identified by Environment Canada for migratory birds to ensure compliance with the *Migratory Birds Convention Act, 1994* (MBCA) and *Migratory Bird Regulations*. The breeding season for migratory birds in this part of the country typically extends from as early as March 1st to as late as September 15th. Should tree removals be required during key breeding bird season, a qualified biologist should inspect those trees to ensure they do not contain nesting birds. It is understood that the MBCA is not limited to cutting woody vegetation, but also applies to topsoil stripping and grubbing activities, as there are ground nesting bird species that are protected under the Act.

6.2.3 Best Management Practices

The design elements are unique and as such, the designer or representative should be part of construction supervision to ensure proper installation and function of the design elements. The designer should confirm materials are appropriate prior to installation. This will ensure the feature functions as

intended. On-site supervision will ensure a rapid response to construction issues. The constructed feature should be deemed stable by the designer, prior to flow introduction.

All works should be isolated from the surrounding natural areas in order to mitigate potential impacts, such as sediment loading. The perimeter of the constructed feature should be stabilized using the prescribed combination of biodegradable erosion control blankets, live staking, and seed. If required, unwatering discharge should be pumped at least 30 m from the existing feature through a filter bag prior to release. The water should be dispersed across the area through straw bales or Filtrexx® SiltSoxx™.

All materials and equipment will be stored and operated in such a manner that prevents any deleterious substances from entering the water. Vehicle and equipment refuelling and/or maintenance will be conducted away from the watercourse and be free of fluid leaks and externally cleaned/degreased to prevent the release of deleterious substances. Machinery should arrive on site in a clean condition (including free of mud/soil/dirt from other locations; including clean wheels/tires/tracks) and should be maintained free of fluid leaks. To reduce the spread of invasive species, equipment should be cleaned before being brought onsite and before leaving site. For guidance in this regard, please refer to the Clean Equipment Protocol for Industry available online: (https://www.ontarioinvasiveplants.ca/wp-content/uploads/2016/07/Clean-Equipment-Protocol_June2016_D3_WEB-1.pdf).

6.2.4 Post-construction Monitoring

A post-construction monitoring program is recommended to assess the performance of the implemented design. Monitoring observations can also be used to determine the need for remedial works, if required. Inspections and monitoring should take place for three full calendar years post-construction. The following monitoring and reporting activities are suggested for the outfall treatment:

- General observations of the outlet treatment should be documented after construction and after the first large flow event to identify any potential areas of erosion concern
- Collect a detailed photographic record of site conditions, including monumented and georeferenced photographs at the treatment location
- A general vegetation survey at the outfall locations in the spring of each year
- A yearly report for the first two years, with a final report at the end of the three-year period
- Monitoring activities should be undertaken by a qualified fluvial geomorphologist
- Sites should be reviewed annually to identify natural variability of the system. Reporting should be provided annually, with a summary report at the end of each year

7 Summary and Recommendations

A hazard assessment completed for both Tributary 1 of Fletcher's Creek and Fletcher's Creek itself determined that a 5 m toe erosion allowance was appropriate given the findings of Soil Engineering Ltd.'s 2020 study and our field observations. This toe erosion allowance was applied wherever the watercourses were within 15 m of the valley wall, in accordance with MOE (2003) guidelines. The final erosion hazard linework for the confined systems is based on the results of a geotechnical slope stability study by Soil Engineers Ltd.

For the short section of channel at the upstream extent of Tributary 1, **Reach FCT-2**, which lacked a valley wall, a meander belt width of 10 m was calculated using a Modified Williams Area Model (1986). Additionally, the meander belt width previously delineated by Parish (2011) along the main branch of Fletcher's Creek (**Reach FC-2**) was refined to be truncated at the toe of slope for Redside Dace habitat delineation. These hazard limits are graphically displayed in **Appendix A**.

For Fletcher's Creek, a desktop assessment identified the ideal location for an outfall location, and an outfall design was provided to mitigate the potential for localized erosion at this site. In terms of potential systemic erosion impacts to Fletcher's Creek resulting from the proposed development, it was determined that given the drainage area of the watercourse relative to the contributing drainage that no exacerbated erosion is expected.

Additionally, the crossing location was also assessed through field verification. The location of the crossing (**Reach FCT-3**) is proposed to cross a poorly defined and vegetation-controlled feature that lacks erosion. A crossing requirement of 2.37 m was recommended however, the minimum crossing width provided below should be considered in conjunction with other requirements.

We trust this report meets your requirements. Should you have any questions please contact the undersigned.

Respectfully submitted,

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Restoration Design Technician

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Appendix A:
Erosion Hazard Mapping

Legend

- Reach Break and ID
- 0.25 m Contour
- Proposed Outfall
- Meander Belt Width
- Meander Belt Width Truncated to Toe of Slope
- 30 m Setback from Meander Belt
- Watercourse
- Drainage Feature
- Crossing Location

Erosion Hazard Assessment

Fletcher's Creek

Mississauga, Ontario

GEO MORPHIX™

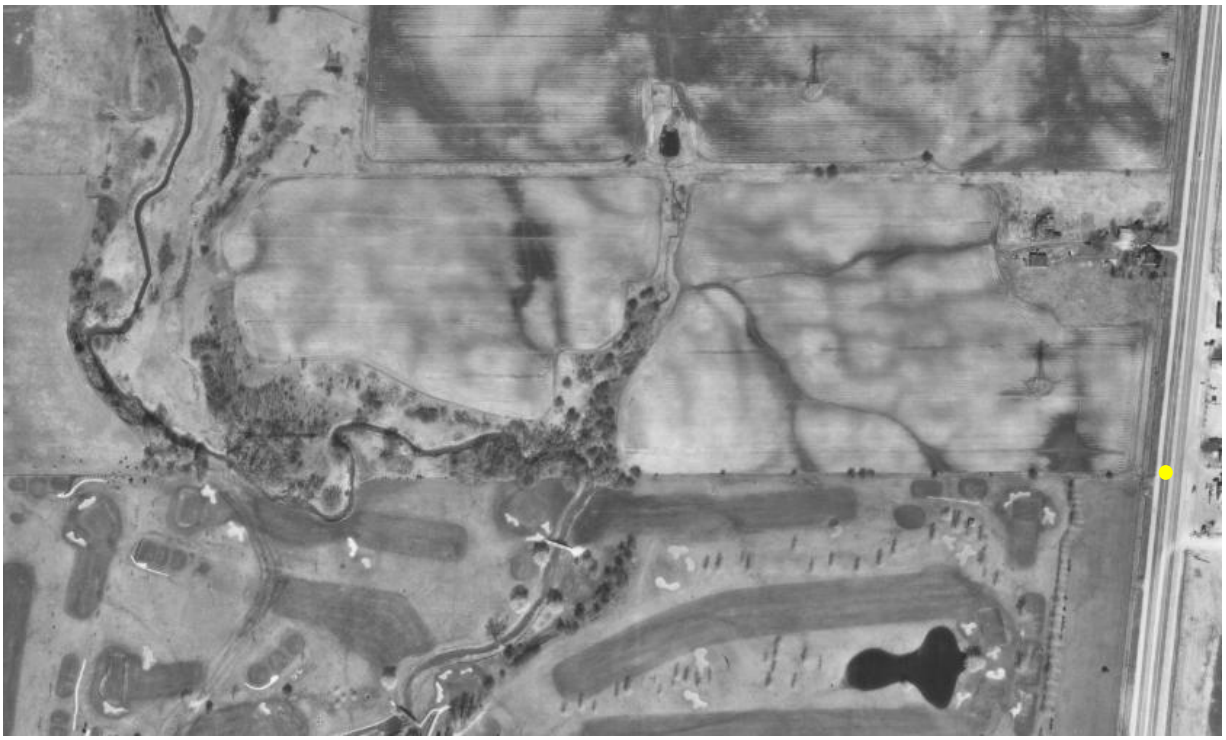
0 40 80
Metres

Imagery: Google Earth. Watercourse: CVC, 2022.
Reach Break and ID: GEO Morphix Ltd., 2018/2025.
0.25 m Contour: Skira & Associates Ltd., 2025. PN25102.
Print Date: September 2025. Drawn By: M.O., R.A., G. U.

Appendix B:
Historical Aerial Imagery

Location: Hurontario Street, Mississauga, Ontario
Year: 1954
Source: National Air Photo Library
Yellow Point: Hurontario Street

Location: Hurontario Street, Mississauga, Ontario
Year: 1977
Source: National Air Photo Library
Yellow Point: Hurontario Street



Location: Hurontario Street, Mississauga, Ontario
Year: 1989
Source: National Air Photo Library
Yellow Point: Hurontario Street

Location: Hurontario Street, Mississauga, Ontario
Year: 2002
Source: Google Earth Pro
Yellow Point: Hurontario Street

Location: Hurontario Street, Mississauga, Ontario
Year: 2002
Source: Google Earth Pro
Yellow Point: Hurontario Street

Location: Hurontario Street, Mississauga, Ontario
Year: 2018
Source: Google Earth Pro
Yellow Point: Hurontario Street

Location: Hurontario Street, Mississauga, Ontario
Year: 2023
Source: Google Earth Pro
Yellow Point: Hurontario Street

Appendix C:
Photographic Field Record

Photo 1 **Outlet Headwall Location,** 7140 Hurontario Street, Mississauga, ON	
	Photograph of staked headwall location facing the direction of the tie-in location.
Photo 2 **Outlet Headwall Location,** 7140 Hurontario Street, Mississauga, ON	
	Photograph of the valley wall behind the headwall location. The slope is well vegetated and showed a lack of erosion at the time of assessment.

Photo 3 **Reach FC-2,** 7140 Hurontario Street, Mississauga, ON	
	Photograph showing the confluence between the historic channel and **Reach FC-2.** The confluence was located at an outer meander bend along **Reach FC-2**.
Photo 4 **Reach FC-2,** 7140 Hurontario Street, Mississauga, ON	
	Photograph of **Reach FC-2** facing upstream. Leaning trees and woody debris jams that impacted flow were common throughout the reach.

Photo 5
Reach FC-2, 7140 Hurontario Street, Mississauga, ON

Coarse material was commonly observed along the channel bed. A suspended armour layer was observed in the channel banks as well. The bank materials consisted of clay, silt materials with some coarse materials due to the suspended armour layer.

Photo 6
Reach FCT-1, 7140 Hurontario Street, Mississauga, ON

Photograph of **Reach FCT-1** facing upstream. The channel was confined and had low flow conditions at the time of assessment. Exposed roots and undercutting were commonly observed. The bed was also eroded into parent material (till) in a localized portion of the reach upstream.

Photo 7
Reach FCT-1, 7140 Hurontario Street, Mississauga, ON

Photograph of **Reach FCT-1**. Active erosion was noted along this reach however; the channel banks were observed to be well vegetated. Some fallen and leaning trees were also observed, indicating some widening along the reach.

Photo 8
Reach FCT-2, 7140 Hurontario Street, Mississauga, ON

Photograph of **Reach FCT-2** facing upstream. The reach was partially confined and had a high density of woody debris and exposed roots.

Photo 9
Reach FCT-2, 7140 Hurontario Street, Mississauga, ON

Photograph of **Reach FCT-2** located at the upstream extent. Note that banks are less defined as the channel moved upstream towards **FCT-3**.

Photo 10
Reach FCT-3, 7140 Hurontario Street, Mississauga, ON

Photograph of **Reach FCT-3** facing upstream towards the pond. Though dry at the time of assessment, much of the reach was comprised of an online wetland containing cattails and phragmites.

Photo 11
FCT-3 Approx. Crossing Location, 7140 Hurontario Street, Mississauga, ON

The proposed water crossing location is situated across **Reach FCT-3.** The location's topography was unconfined, flat, and the feature was poorly defined. Herbaceous plants and tall grasses dominated the area.

Appendix D:
Field Observations

GEO MORPHIX™

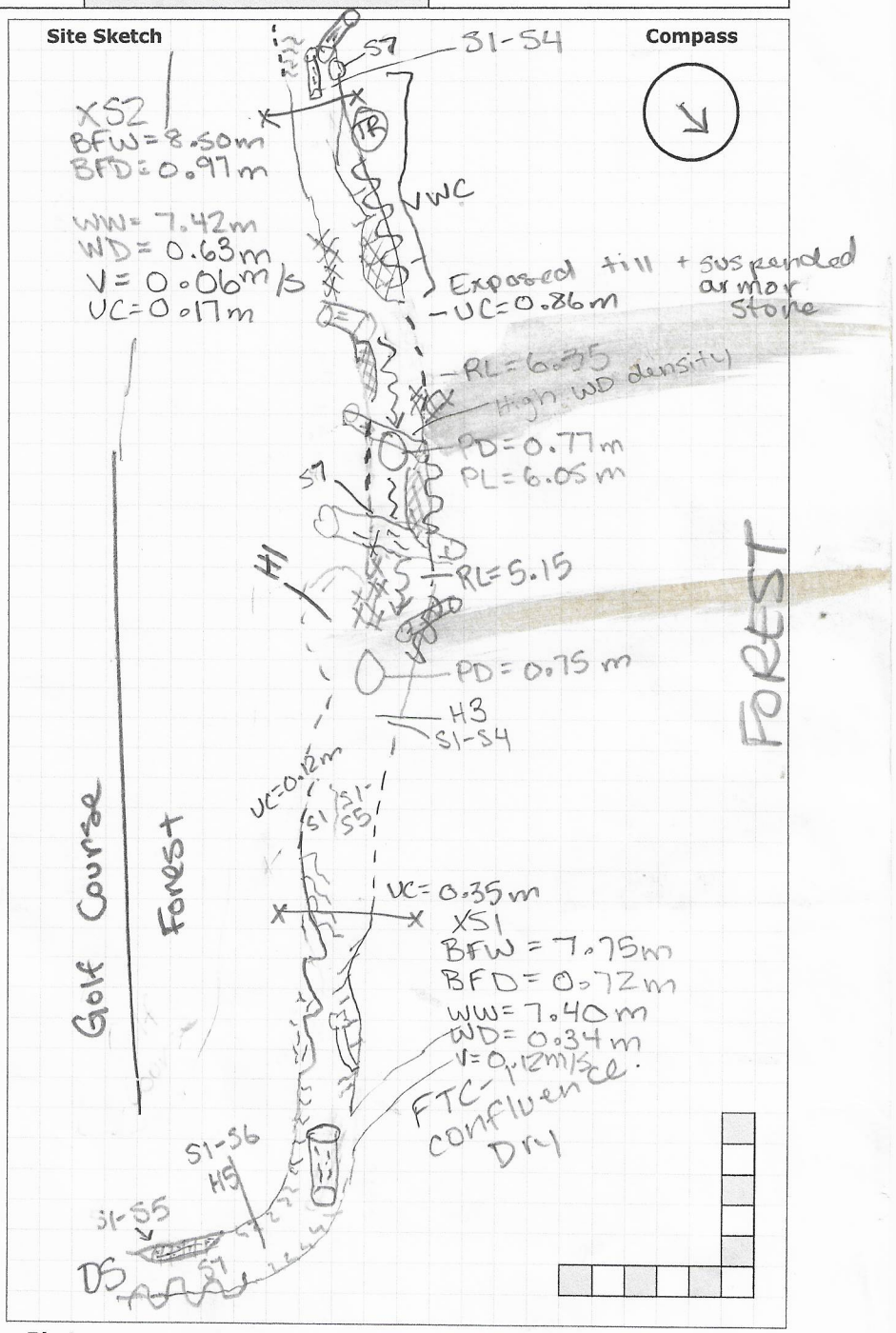
General Site Characteristics

Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	9:00	**Reach:**	FC-2
Weather:	24°C, Sunny	**Location:**	Derry Crest Rd.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		mass wst. slump
	Bank stabilization		exposed roots
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch



Flow Type			
H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
H9	Free fall	**H9A**	Dissipates below free fall

Substrate			
S1	Silt	**S6**	Small boulder
S2	Sand	**S7**	Large boulder
S3	Gravel	**S8**	Bimodal
S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other			
BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Photos:

Notes:

Bank Materials = Clay, silt, sand, cobbles

GEO MORPHIX

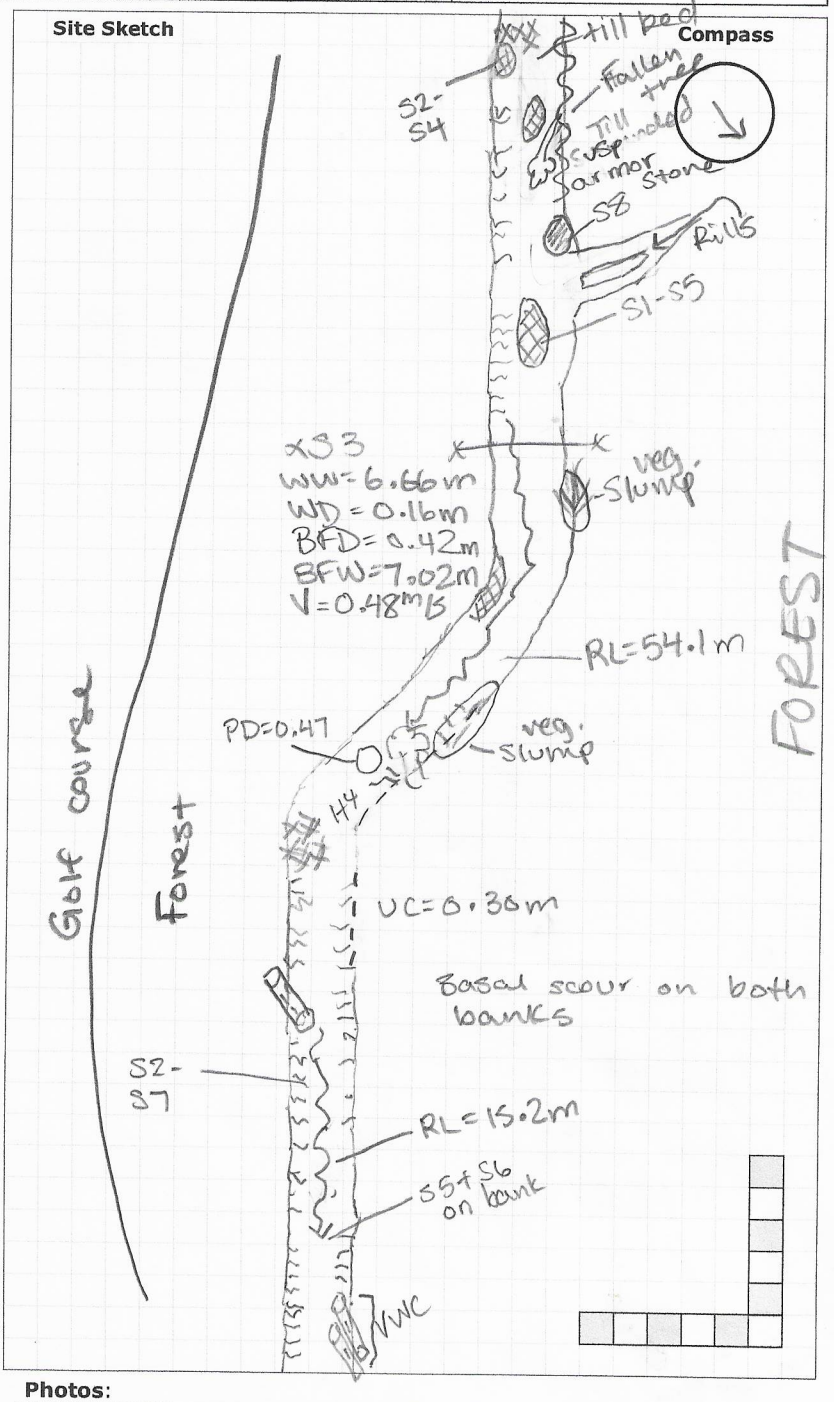
General Site Characteristics

Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	9.30	**Reach:**	FC-2
Weather:	24°C, Sunny	**Location:**	Derry Crest Dr.
Field Staff:	NH, CM	**Watershed/Subwatershed:**	

Features	Monitoring
Reach break	Long-profile
Station location	Monumented XS
Cross-section	Monumented photo
Flow direction	Monumented photo direction
Riffle	Sediment sampling
Pool	Erosion pins
Sediment bar	Scour chains
Eroded bank/slope	**Additional Symbols**
Undercut bank	
Bank stabilization	
Leaning tree	
Fence	
Culvert/outfall	
Swamp/wetland	
Grasses	
Tree	
Instream log/tree	
Woody debris	
Beaver dam	
Vegetated island	

Site Sketch



Photos:

Notes:

Flow Type

H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
H9	Free fall	**H9A**	Dissipates below free fall

Substrate

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S4	Small cobble	**S9**	Bedrock/till
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Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

GEO MORPHIX™

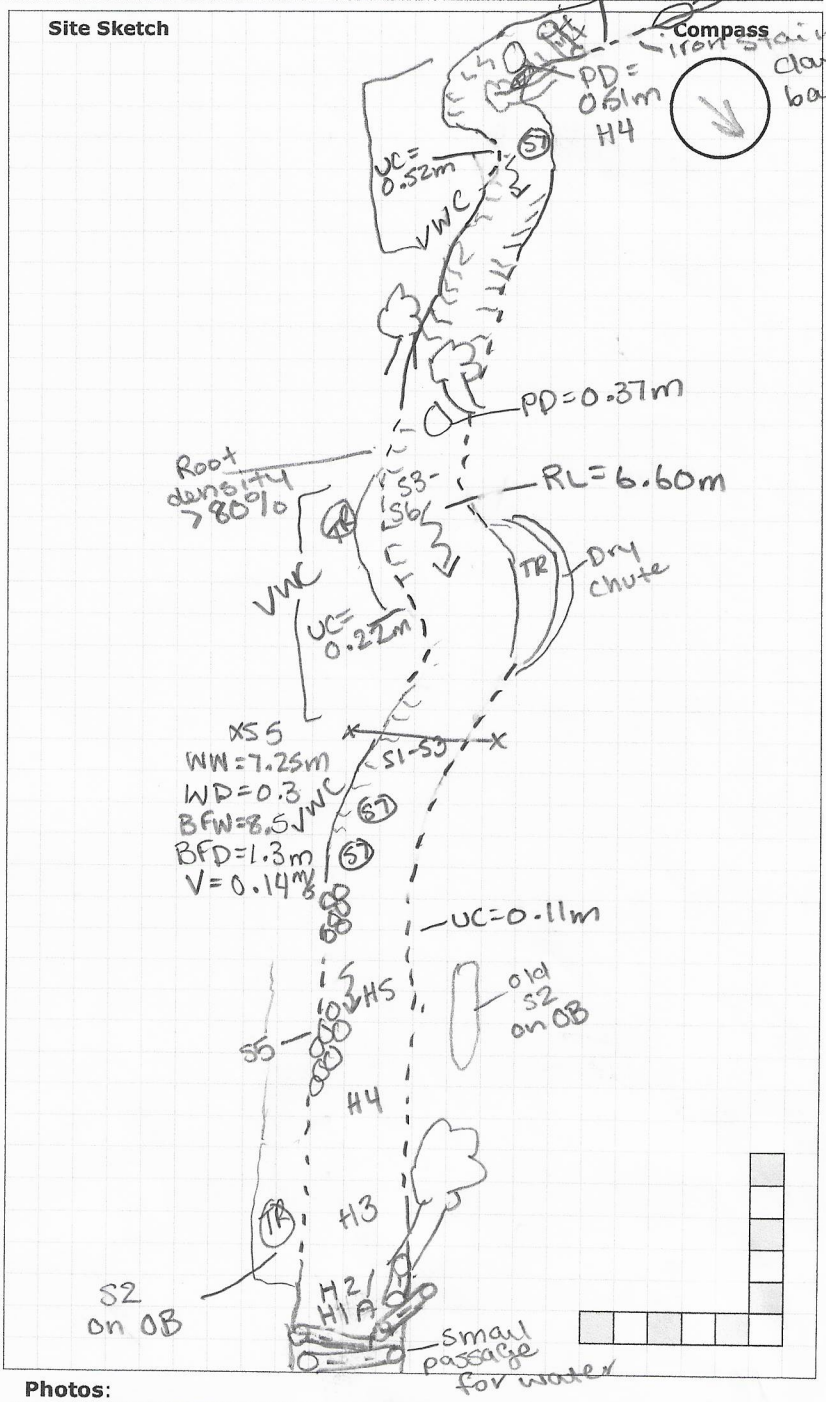
General Site Characteristics

Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	10:00	**Reach:**	FC-2
Weather:	24°C, Sunny	**Location:**	Dreamy UC=0.82m St Dr.
Field Staff:	NH, CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch



Photos:

Notes:

Flow Type

H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
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Substrate

S1	Silt	**S6**	Small boulder
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S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

GEO MORPHIX™

General Site Characteristics

Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	10:30	**Reach:**	FC-2
Weather:	24°C, Sunny	**Location:**	Derry Crest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Flow Type

H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
H9	Free fall	**H9A**	Dissipates below free fall

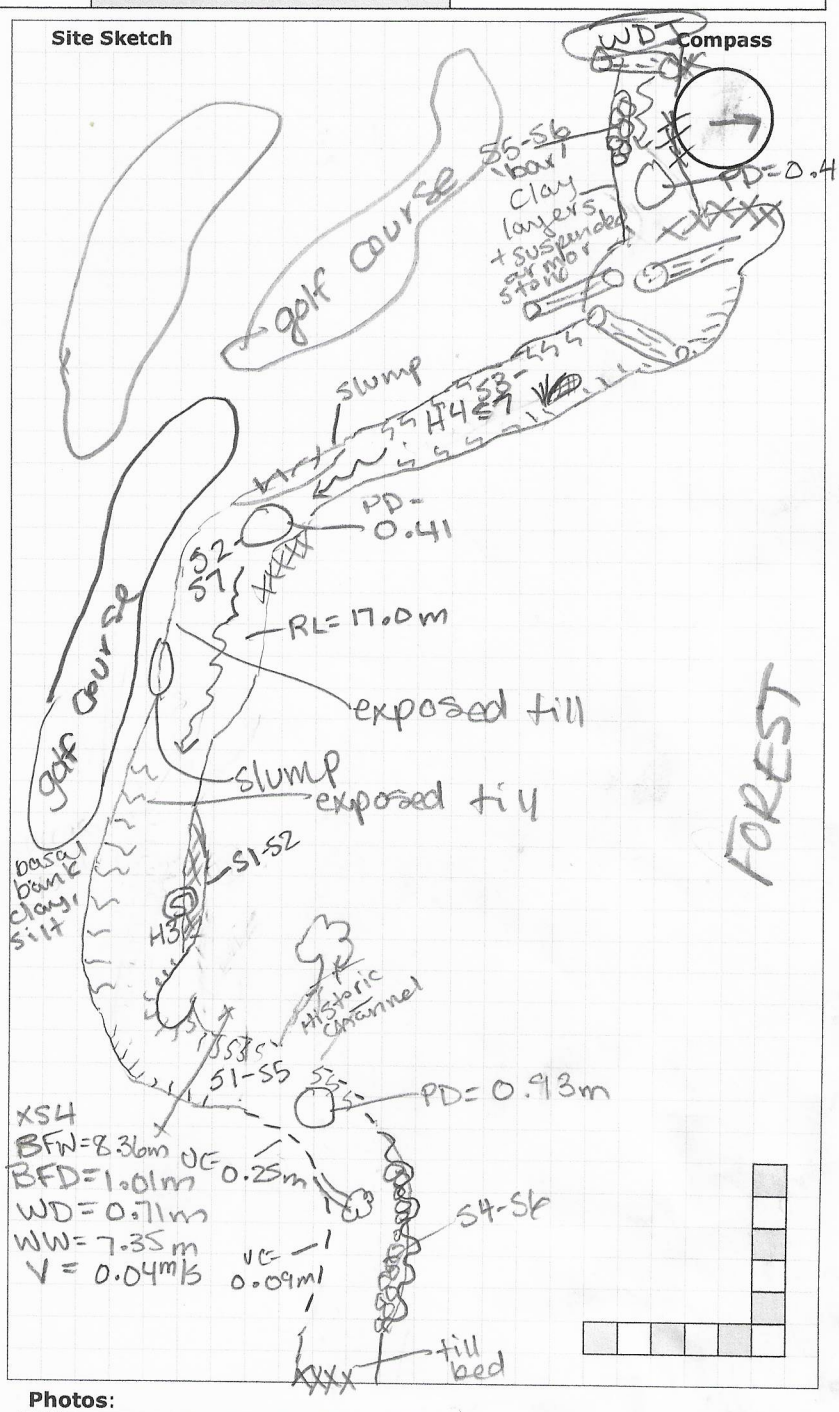
Substrate

S1	Silt	**S6**	Small boulder
S2	Sand	**S7**	Large boulder
S3	Gravel	**S8**	Bimodal
S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Site Sketch



Photos:

Notes:

GEO MORPHIX™

General Site Characteristics

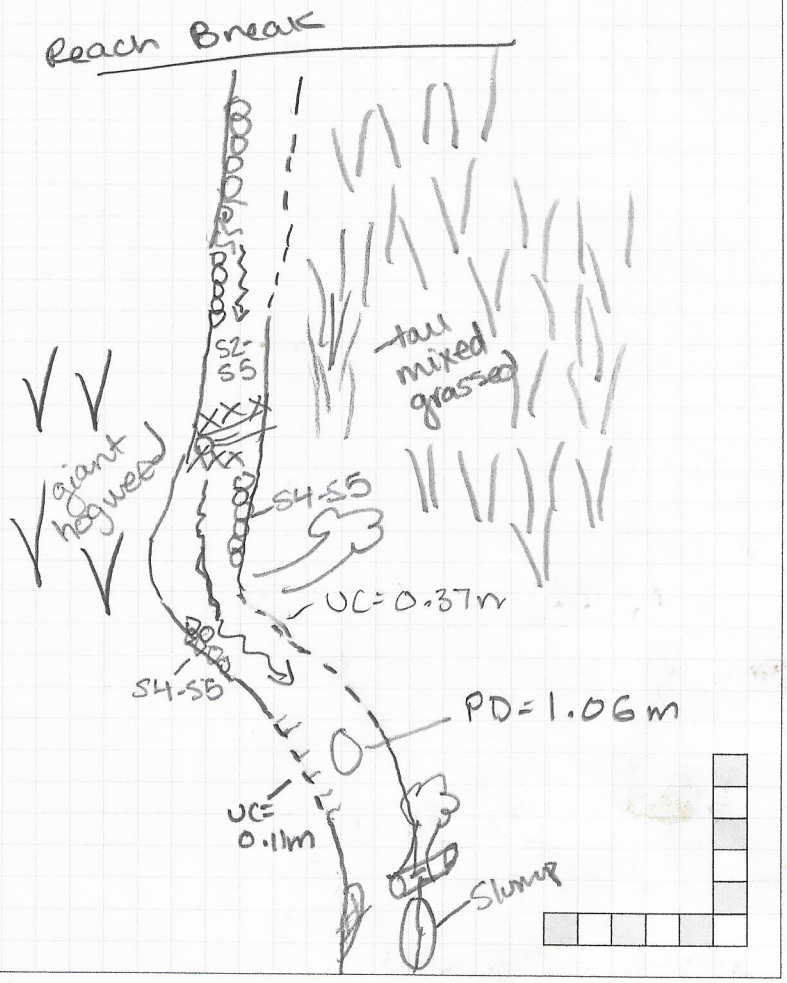
Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletchers Creek
Time:	11:00	**Reach:**	FC-2
Weather:	24°C, Sunny	**Location:**	Derry/Crest Dr.
Field Staff:	NH, CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch

Compass



Flow Type

H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
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Substrate

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S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Photos:

Notes:

GEO MORPHIX™

Rapid Stream Assessment Technique Project Number: 2502

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	11:18	**Reach:**	FC-2
Weather:	Sun 25	**Location:**	Derrycrest Rd
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Category	Poor	Fair	Good	Excellent
Channel Stability	• < 50% of bank network stable • Recent bank sloughing, slumping or failure frequently observed	• 50-70% of bank network stable • Recent signs of bank sloughing, slumping or failure fairly common	• 71-80% of bank network stable • Infrequent signs of bank sloughing, slumping or failure	• > 80% of bank network stable • No evidence of bank sloughing, slumping or failure
	• Stream bend areas highly unstable • Outer bank height 1.2 m above stream bank (2.1 m above stream bank for large mainstem areas) • Bank overhang > 0.8-1.0 m	• Stream bend areas unstable • Outer bank height 0.9-1.2 m above stream bank (1.5-2.1 m above stream bank for large mainstem areas) • Bank overhang 0.8-0.9m	• Stream bend areas stable • Outer bank height 0.6-0.9 m above stream bank (1.2-1.5 m above stream bank for large mainstem areas) • Bank overhang 0.6-0.8 m	• Stream bend areas very stable • Height < 0.6 m above stream (< 1.2 m above stream bank for large mainstem areas) • Bank overhang < 0.6 m
	• Young exposed tree roots abundant • > 6 recent large tree falls per stream mile	• Young exposed tree roots common • 4-5 recent large tree falls per stream mile	• Exposed tree roots predominantly old and large, smaller young roots scarce • 2-3 recent large tree falls per stream mile	• Exposed tree roots old, large and woody • Generally 0-1 recent large tree falls per stream mile
	• Bottom 1/3 of bank is highly erodible material • Plant/soil matrix severely compromised	• Bottom 1/3 of bank is generally highly erodible material • Plant/soil matrix compromised	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material - clay/silt	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material
	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally V- or U-shaped	• Channel cross-section is generally V- or U-shaped
Point range	☐ 0 ☐ 1 ☐ 2	☑ 3 ☐ 4 ☐ 5	☐ 6 ☐ 7 ☐ 8	☐ 9 ☐ 10 ☐ 11
Channel Scouring/ Sediment Deposition	• > 75% embedded (> 85% embedded for large mainstem areas)	• 50-75% embedded (60-85% embedded for large mainstem areas)	• 25-49% embedded (35-59% embedded for large mainstem areas)	• Riffle embeddedness < 25% sand-silt (< 35% embedded for large mainstem areas)
	• Few, if any, deep pools • Pool substrate composition >81% sand-silt	• Low to moderate number of deep pools • Pool substrate composition 60-80% sand-silt	• Moderate number of deep pools • Pool substrate composition 30-59% sand-silt	• High number of deep pools (> 61 cm deep) (> 122 cm deep for large mainstem areas) • Pool substrate composition <30% sand-silt
	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits uncommon	• Streambed streak marks and/or "banana"-shaped sediment deposits absent
	• Fresh, large sand deposits very common in channel • Moderate to heavy sand deposition along major portion of overbank area	• Fresh, large sand deposits common in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits uncommon in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits rare or absent from channel • No evidence of fresh sediment deposition on overbank
	• Point bars present at most stream bends, moderate to large and unstable with high amount of fresh sand	• Point bars common, moderate to large and unstable with high amount of fresh sand	• Point bars small and stable, well-vegetated and/or armoured with little or no fresh sand	• Point bars few, small and stable, well-vegetated and/or armoured with little or no fresh sand
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☐ 5 ☐ 6	☑ 7 ☐ 8

Version #2
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: CM Completed by: NH

GEO MORPHIX™

Date:	2025-07-18	PN:	25102	Location:	

Category	Poor	Fair	Good	Excellent
Physical Instream Habitat	• Wetted perimeter < 40% of bottom channel width (< 45% for large mainstem areas)	• Wetted perimeter 40-60% of bottom channel width (45-65% for large mainstem areas)	• Wetted perimeter 61-85% of bottom channel width (66-90% for large mainstem areas)	• Wetted perimeter > 85% of bottom channel width (> 90% for large mainstem areas)
	• Dominated by one habitat type (usually runs) and by one velocity and depth condition (slow and shallow) (for large mainstem areas, few riffles present, runs and pools dominant, velocity and depth diversity low)	• Few pools present, riffles and runs dominant. • Velocity and depth generally slow and shallow (for large mainstem areas, runs and pools dominant, velocity and depth diversity intermediate)	• Good mix between riffles, runs and pools • Relatively diverse velocity and depth of flow	• Riffles, runs and pool habitat present • Diverse velocity and depth of flow present (i.e., slow, fast, shallow and deep water)
	• Riffle substrate composition: predominantly gravel with high amount of sand • < 5% cobble	• Riffle substrate composition: predominantly small cobble, gravel and sand • 5-24% cobble	• Riffle substrate composition: good mix of gravel, cobble, and rubble material • 25-49% cobble	• Riffle substrate composition: cobble, gravel, rubble, boulder mix with little sand • > 50% cobble
	• Riffle depth < 10 cm for large mainstem areas	• Riffle depth 10-15 cm for large mainstem areas	• Riffle depth 15-20 cm for large mainstem areas	• Riffle depth > 20 cm for large mainstem areas
	• Large pools generally < 30 cm deep (< 61 cm for large mainstem areas) and devoid of overhead cover/structure	• Large pools generally 30-46 cm deep (61-91 cm for large mainstem areas) with little or no overhead cover/structure	• Large pools generally 46-61 cm deep (91-122 cm for large mainstem areas) with some overhead cover/structure	• Large pools generally > 61 cm deep (> 122 cm for large mainstem areas) with good overhead cover/structure
	• Extensive channel alteration and/or point bar formation/enlargement	• Moderate amount of channel alteration and/or moderate increase in point bar formation/enlargement	• Slight amount of channel alteration and/or slight increase in point bar formation/enlargement	• No channel alteration or significant point bar formation/enlargement
	• Riffle/Pool ratio 0.49:1 ; ≥1.51:1	• Riffle/Pool ratio 0.5-0.69:1 ; 1.31-1.5:1	• Riffle/Pool ratio 0.7-0.89:1 ; 1.11-1.3:1	• Riffle/Pool ratio 0.9-1.1:1
	• Summer afternoon water temperature > 27°C	• Summer afternoon water temperature 24-27°C	• Summer afternoon water temperature 20-24°C	• Summer afternoon water temperature < 20°C
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☐ 5 ☐ 6	☒ 7 ☐ 8
Water Quality	• Substrate fouling level: High (> 50%)	• Substrate fouling level: Moderate (21-50%)	• Substrate fouling level: Very light (11-20%)	• Substrate fouling level: Rock underside (0-10%)
	• Brown colour • TDS: > 150 mg/L	• Grey colour • TDS: 101-150 mg/L	• Slightly grey colour • TDS: 50-100 mg/L	• Clear flow • TDS: < 50 mg/L
	• Objects visible to depth < 0.15m below surface	• Objects visible to depth 0.15-0.5m below surface	• Objects visible to depth 0.5-1.0m below surface	• Objects visible to depth > 1.0m below surface
	• Moderate to strong organic odour	• Slight to moderate organic odour	• Slight organic odour	• No odour
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☐ 5 ☒ 6	☐ 7 ☐ 8
Riparian Habitat Conditions	• Narrow riparian area of mostly non-woody vegetation	• Riparian area predominantly wooded but with major localized gaps	• Forested buffer generally > 31 m wide along major portion of both banks	• Wide (> 60 m) mature forested buffer along both banks
	• Canopy coverage: <50% shading (30% for large mainstem areas)	• Canopy coverage: 50-60% shading (30-44% for large mainstem areas)	• Canopy coverage: 60-79% shading (45-59% for large mainstem areas)	• Canopy coverage: >80% shading (> 60% for large mainstem areas)
Point range	☐ 0 ☐ 1	☐ 2 ☐ 3	☒ 4 ☐ 5	☐ 6 ☐ 7

Total overall score (0–42) = 27	Poor (<13)	Fair (13-24)	Good (25-34)	Excellent (>35)

Version #2
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: MH

GEO MORPHIX

Reach Characteristics Project Number: 25102

Date:	2025-07-18	**Field Staff:**	CM NH	**Watershed/Subwatershed:**	
Time:	9:00	**Stream:**	Fletcher's Creek	**UTM (Upstream):**	
Weather:	Sun 25°	**Reach:**	FC-2	**UTM (Downstream):**	

Land Use (Table 1): 1,8 | **Valley Type** (Table 2): 3 | **Channel Type** (Table 3): 8 | **Channel Zone** (Table 4): 2 | **Flow Type** (Table 5): 1 | ☐ Evidence of Groundwater Location:_____ Photo:_____

Riparian Vegetation

Dominant Type (Table 6): 1 | **Encroachment** (Table 7): 1

Coverage	Channel Widths	Age (yrs)
☐ None	☐ 1 - 4	☐ Immature (<5)
☒ Fragmented	☒ 4 - 10	☐ Established (5-30)
☐ Continuous	☐ > 10	☒ Mature (>30)

Aquatic & Instream Vegetation

Type (Table 8): 1 | **Reach Coverage %**: 0.5

Woody Debris	WD Density
☒ In Cutbank	☐ Low
☒ In Channel	☐ Mod
☐ Not Present	☒ High

WDJ/50m: 2

Water Quality

Odour (Table 16): 1 | **Turbidity** (Table 17): 1

Channel Characteristics

Sinuosity Type (Table 9): 2 | **Sinuosity Degree** (Table 10): 2

Gradient (Table 11): 1 | **# of Channels** (Table 12): 1

Entrenchment (Table 13): 1 | **Bank Failure** (Table 14): 2,5,6

Down's Model (Table 15): E/U | **Bankfull Indicators** (Table 18): 1,3,5 7

Sed Sorting (Table 20): MoD | **Sediment Transport Observed?** ☐ Yes ☒ No ☐ Not Visible

Transport Mode (Table 21): 1 | **% of Bed Active**: 0

Geomorphic Units (Table 22): 4-8 | **Mass Movement** (Table 23): 1,3

Riffle-Pool Spacing (m): | **% Riffles**: 35 | **% Pools**: 35

Bank Angle	Bank Erosion
☐ 0 - 30	☐ < 5%
☒ 30 - 60	☐ 5 - 30%
☒ 60 - 90	☐ 30 - 60%
☐ Undercut	☒ 60 - 100%

(Table 19)

	Clay/Silt	Sand	Gravel	Cobble	Boulder	Parent	Rootlets
Bank	☒	☒	☐	☐	☐	☐	☒
Riffle	☐	☐	☒	☒	☒	☐	☐
Pool	☒	☒	☒	☐	☐	☐	☐
Bed (if no riffle-pool morphology)	☐	☐	☐	☐	☐	☐	☐

Bankfull Width (m)	7.25	8.5	8.36	**Wetted Width (m)**	7.40	7.25	7.35
Bankfull Depth (m)	0.72	1.3	1.01	**Wetted Depth (m)**	0.34	0.3	0.71
Undercuts (m)	0.35	0.86	0.11	**Velocity (m/s)**	0.12	0.14	0.04
Pool Depth (m)	0.75	0.93	0.42	**Velocity Estimate Method**	WB	WB	WB
Riffle Length (m)	6.35	15.2	17.0	**Meander Amplitude (m)**	/	/	/

Notes:

- Channel flows through forest/golf course landscape. Buffer width varies. Trees + shrubs dominant
- VWC observed on both banks throughout reach
- Irregular meanders present but infrequent. Meander amplitude could not be accurately measured
- Cobbles dominant bed substrate. Scattered boulders present.
- Undercutting/basal scour present on both banks. Inside of bend undercut.
- Fallen trees common but most were dead and likely not recent falls
- Mass rotational slides/slumps observed on both banks.
- Minnows observed throughout reach

Photos:

- Large WDJ observed causing back watering US of golf course
- Extensive undercuts on both banks at tie in confluence
- Till bed observed US and DS on tie in confluence

Version #4
Last edited: 04/04/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: CM

GEO MORPHIX™

Rapid Geomorphic Assessment **Project Number:** 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	11:28	**Reach:**	FC-2
Weather:	Sun 24	**Location:**	Derrycrest Dr, On
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Process	No.	Geomorphological Indicator	Present? Yes	Present? No	Factor Value
Evidence of Aggradation (AI)	1	Lobate bar		✓	2/7
	2	Coarse materials in riffles embedded		✓	
	3	Siltation in pools	✓		
	4	Medial bars – 2 @ course materials, 1 @ vfg	✓		
	5	Accretion on point bars		✓	
	6	Poor longitudinal sorting of bed materials		✓	
	7	Deposition in the overbank zone – 1 area US of WDJ		✓	
		Sum of indices =	2	5	0.29
Evidence of Degradation (DI)	1	Exposed bridge footing(s) N/A	—	—	2/6
	2	Exposed sanitary / storm sewer / pipeline / etc.		✓	
	3	Elevated storm sewer outfall(s) N/A	—		
	4	Undermined gabion baskets / concrete aprons / etc. N/A	—	—	
	5	Scour pools downstream of culverts / storm sewer outlets N/A	—	—	
	6	Cut face on bar forms		✓	
	7	Head cutting due to knickpoint migration		✓	
	8	Terrace cut through older bar material	✓		
	9	Suspended armour layer visible in bank	✓		
	10	Channel worn into undisturbed overburden / bedrock – 1 area		✓	
		Sum of indices =	2	4	0.33
Evidence of Widening (WI)	1	Fallen / leaning trees / fence posts / etc.	✓		6/8
	2	Occurrence of large organic debris	✓		
	3	Exposed tree roots	✓		
	4	Basal scour on inside meander bends	✓		
	5	Basal scour on both sides of channel through riffle	✓		
	6	Outflanked gabion baskets / concrete walls / etc. N/A	—		
	7	Length of basal scour >50% through subject reach	✓		
	8	Exposed length of previously buried pipe / cable / etc. – tile @ golf course		✓	
	9	Fracture lines along top of bank		✓	
	10	Exposed building foundation N/A	—	—	
		Sum of indices =	6	2	0.75
Evidence of Planimetric Form Adjustment (PI)	1	Formation of chute(s) – 1 chute, dry		✓	0/7
	2	Single thread channel to multiple channel		✓	
	3	Evolution of pool-riffle form to low bed relief form		✓	
	4	Cut-off channel(s)		✓	
	5	Formation of island(s)		✓	
	6	Thalweg alignment out of phase with meander form		✓	
	7	Bar forms poorly formed / reworked / removed		✓	
		Sum of indices =	0	0	0

Notes: High density of fallen trees + exposed roots
VWC on both banks
Slumping more common in US extent

Stability Index (SI) = (AI+DI+WI+PI)/4 = 0.34

In Regime	In Transition/Stress	In Adjustment
☐ 0.00 - 0.20	☒ 0.21 - 0.40	☐ 0.41

Version #3
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: CM Completed by: NH

GEO MORPHIX™

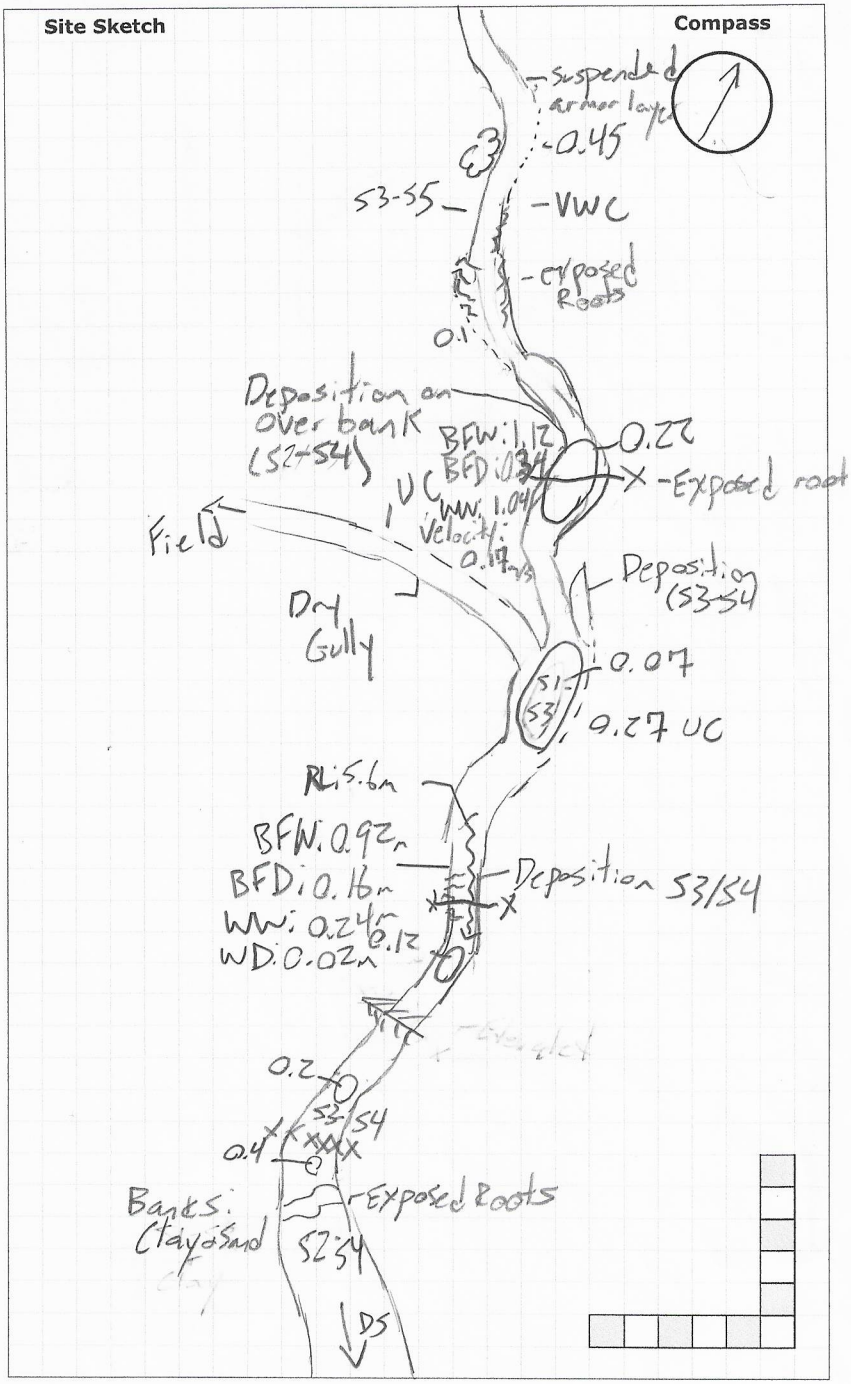
General Site Characteristics

Project Number: 25102

Date:	2025-07-16	**Stream:**	Fletcher's Creek
Time:	12:45	**Reach:**	FTC-1
Weather:	24°C, Sunny	**Location:**	Derry Crest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch



Flow Type			
H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
H9	Free fall	**H9A**	Dissipates below free fall

Substrate			
S1	Silt	**S6**	Small boulder
S2	Sand	**S7**	Large boulder
S3	Gravel	**S8**	Bimodal
S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other			
BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Photos:

Notes:

GEO MORPHIX™

General Site Characteristics

Project Number: 25102

Date:	2025-07-16	**Stream:**	Fletcher's Creek
Time:	13:55	**Reach:**	FTC-1
Weather:	24°C, Sunny	**Location:**	Deery Crest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
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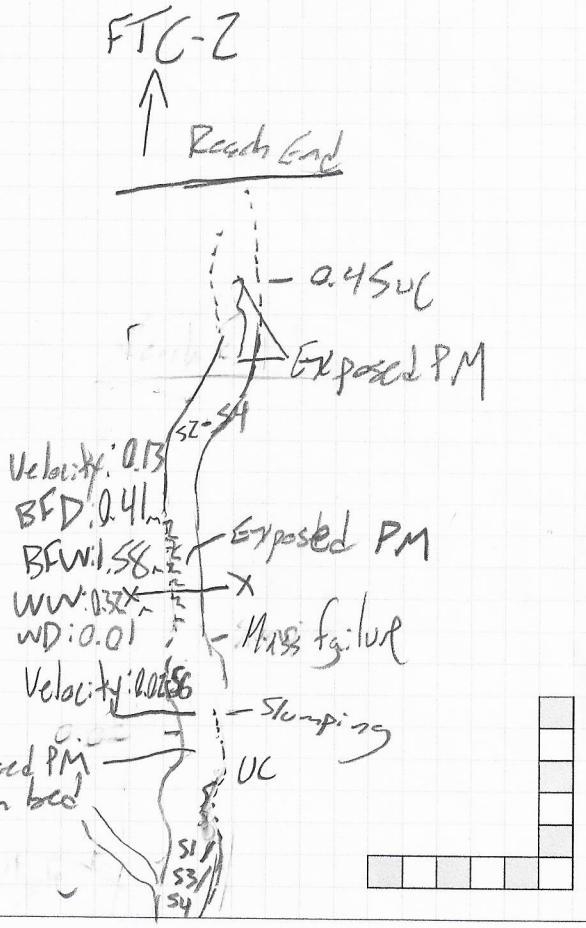
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch

Compass



Flow Type

H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
H9	Free fall	**H9A**	Dissipates below free fall

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S2	Sand	**S7**	Large boulder
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S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Photos:

Notes:

GEO MORPHIX™

Rapid Stream Assessment Technique Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	12:45	**Reach:**	FTC-1
Weather:	Sun 25°	**Location:**	Derrycrest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Category	Poor	Fair	Good	Excellent
Channel Stability	• < 50% of bank network stable • Recent bank sloughing, slumping or failure frequently observed	• 50-70% of bank network stable • Recent signs of bank sloughing, slumping or failure fairly common	• 71-80% of bank network stable • Infrequent signs of bank sloughing, slumping or failure	• > 80% of bank network stable • No evidence of bank sloughing, slumping or failure
	• Stream bend areas highly unstable • Outer bank height 1.2 m above stream bank (2.1 m above stream bank for large mainstem areas) • Bank overhang > 0.8-1.0 m	• Stream bend areas unstable • Outer bank height 0.9-1.2 m above stream bank (1.5-2.1 m above stream bank for large mainstem areas) • Bank overhang 0.8-0.9m	• Stream bend areas stable • Outer bank height 0.6-0.9 m above stream bank (1.2-1.5 m above stream bank for large mainstem areas) • Bank overhang 0.6-0.8 m	• Stream bend areas very stable • Height < 0.6 m above stream (< 1.2 m above stream bank for large mainstem areas) • Bank overhang < 0.6 m
	• Young exposed tree roots abundant • > 6 recent large tree falls per stream mile	• Young exposed tree roots common • 4-5 recent large tree falls per stream mile	• Exposed tree roots predominantly old and large, smaller young roots scarce • 2-3 recent large tree falls per stream mile	• Exposed tree roots old, large and woody • Generally 0-1 recent large tree falls per stream mile
	• Bottom 1/3 of bank is highly erodible material • Plant/soil matrix severely compromised	• Bottom 1/3 of bank is generally highly erodible material • Plant/soil matrix compromised	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material
	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally V- or U-shaped	• Channel cross-section is generally V- or U-shaped
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☒ 4 ☐ 5	☐ 6 ☐ 7 ☐ 8	☐ 9 ☐ 10 ☐ 11
Channel Scouring/ Sediment Deposition	• > 75% embedded (> 85% embedded for large mainstem areas)	• 50-75% embedded (60-85% embedded for large mainstem areas)	• 25-49% embedded (35-59% embedded for large mainstem areas)	• Riffle embeddedness < 25% sand-silt (< 35% embedded for large mainstem areas)
	• Few, if any, deep pools • Pool substrate composition >81% sand-silt	• Low to moderate number of deep pools • Pool substrate composition 60-80% sand-silt	• Moderate number of deep pools • Pool substrate composition 30-59% sand-silt	• High number of deep pools (> 61 cm deep) (> 122 cm deep for large mainstem areas) • Pool substrate composition <30% sand-silt
	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits uncommon	• Streambed streak marks and/or "banana"-shaped sediment deposits absent
	• Fresh, large sand deposits very common in channel • Moderate to heavy sand deposition along major portion of overbank area	• Fresh, large sand deposits common in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits uncommon in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits rare or absent from channel • No evidence of fresh sediment deposition on overbank
	• Point bars present at most stream bends, moderate to large and unstable with high amount of fresh sand	• Point bars common, moderate to large and unstable with high amount of fresh sand	• Point bars small and stable, well-vegetated and/or armoured with little or no fresh sand	• Point bars few, small and stable, well-vegetated and/or armoured with little or no fresh sand
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☐ 5 ☒ 6	☐ 7 ☐ 8

Version #2
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: CM Completed by: NH

GEO MORPHIX™

Date:	2025-07-18	PN:	25102	Location:	

Category	Poor	Fair	Good	Excellent
Physical Instream Habitat	• Wetted perimeter < 40% of bottom channel width (< 45% for large mainstem areas)	• Wetted perimeter 40-60% of bottom channel width (45-65% for large mainstem areas)	• Wetted perimeter 61-85% of bottom channel width (66-90% for large mainstem areas)	• Wetted perimeter > 85% of bottom channel width (> 90% for large mainstem areas)
	• Dominated by one habitat type (usually runs) and by one velocity and depth condition (slow and shallow) (for large mainstem areas, few riffles present, runs and pools dominant, velocity and depth diversity low)	• Few pools present, riffles and runs dominant. • Velocity and depth generally slow and shallow (for large mainstem areas, runs and pools dominant, velocity and depth diversity intermediate)	• Good mix between riffles, runs and pools • Relatively diverse velocity and depth of flow	• Riffles, runs and pool habitat present • Diverse velocity and depth of flow present (i.e., slow, fast, shallow and deep water)
	• Riffle substrate composition: predominantly gravel with high amount of sand • < 5% cobble	• Riffle substrate composition: predominantly small cobble, gravel and sand • 5-24% cobble	• Riffle substrate composition: good mix of gravel, cobble, and rubble material • 25-49% cobble	• Riffle substrate composition: cobble, gravel, rubble, boulder mix with little sand • > 50% cobble
	• Riffle depth < 10 cm for large mainstem areas	• Riffle depth 10-15 cm for large mainstem areas	• Riffle depth 15-20 cm for large mainstem areas	• Riffle depth > 20 cm for large mainstem areas
	• Large pools generally < 30 cm deep (< 61 cm for large mainstem areas) and devoid of overhead cover/structure	• Large pools generally 30-46 cm deep (61-91 cm for large mainstem areas) with little or no overhead cover/structure	• Large pools generally 46-61 cm deep (91-122 cm for large mainstem areas) with some overhead cover/structure	• Large pools generally > 61 cm deep (> 122 cm for large mainstem areas) with good overhead cover/structure
	• Extensive channel alteration and/or point bar formation/enlargement	• Moderate amount of channel alteration and/or moderate increase in point bar formation/enlargement	• Slight amount of channel alteration and/or slight increase in point bar formation/enlargement	• No channel alteration or significant point bar formation/enlargement
	• Riffle/Pool ratio 0.49:1 ; ≥1.51:1	• Riffle/Pool ratio 0.5-0.69:1 ; 1.31-1.5:1	• Riffle/Pool ratio 0.7-0.89:1 ; 1.11-1.3:1	• Riffle/Pool ratio 0.9-1.1:1
	• Summer afternoon water temperature > 27°C	• Summer afternoon water temperature 24-27°C	• Summer afternoon water temperature 20-24°C	• Summer afternoon water temperature < 20°C
Point range	☐ 0 ☐ 1 ☐ 2	☒ 3 ☐ 4	☐ 5 ☐ 6	☐ 7 ☐ 8
Water Quality	• Substrate fouling level: High (> 50%)	• Substrate fouling level: Moderate (21-50%)	• Substrate fouling level: Very light (11-20%)	• Substrate fouling level: Rock underside (0-10%)
	• Brown colour • TDS: > 150 mg/L	• Grey colour • TDS: 101-150 mg/L	• Slightly grey colour • TDS: 50-100 mg/L	• Clear flow • TDS: < 50 mg/L
	• Objects visible to depth < 0.15m below surface	• Objects visible to depth 0.15-0.5m below surface	• Objects visible to depth 0.5-1.0m below surface	• Objects visible to depth > 1.0m below surface
	• Moderate to strong organic odour	• Slight to moderate organic odour	• Slight organic odour	• No odour
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☐ 5 ☐ 6	☐ 7 ☒ 8
Riparian Habitat Conditions	• Narrow riparian area of mostly non-woody vegetation	• Riparian area predominantly wooded but with major localized gaps	• Forested buffer generally > 31 m wide along major portion of both banks	• Wide (> 60 m) mature forested buffer along both banks
	• Canopy coverage: <50% shading (30% for large mainstem areas)	• Canopy coverage: 50-60% shading (30-44% for large mainstem areas)	• Canopy coverage: 60-79% shading (45-59% for large mainstem areas)	• Canopy coverage: >80% shading (> 60% for large mainstem areas)
Point range	☐ 0 ☐ 1	☐ 2 ☐ 3	☐ 4 ☐ 5	☒ 6 ☐ 7

Total overall score (0–42) = 27	Poor (<13)	Fair (13-24)	Good (25-34)	Excellent (>35)

Version #2
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: CM

GEO MORPHIX

Reach Characteristics Project Number: 25102

Date:	2025-07-18	**Field Staff:**	NH CM	**Watershed/Subwatershed:**	
Time:	12:45	**Stream:**	Fletcher's Creek	**UTM (Upstream):**	
Weather:	Sun 25	**Reach:**	FCT-1	**UTM (Downstream):**	

Land Use (Table 1): 1,3,6 | **Valley Type** (Table 2): 2 | **Channel Type** (Table 3): 6 | **Channel Zone** (Table 4): 2 | **Flow Type** (Table 5): 1 | ☐ Evidence of Groundwater Location:_____ Photo:_____

Riparian Vegetation

Dominant Type (Table 6): 1
Encroachment (Table 7): 2

Coverage	Channel Widths	Age (yrs)
☐ None	☐ 1 - 4	☐ Immature (<5)
☐ Fragmented	☐ 4 - 10	☐ Established (5-30)
☒ Continuous	☒ > 10	☒ Mature (>30)

Aquatic & Instream Vegetation

Type (Table 8): N/A
Reach Coverage %: 0

Woody Debris	WD Density
☒ In Cutbank	☐ Low
☒ In Channel	☒ Mod
☐ Not Present	☐ High

WDJ/50m: 1

Water Quality

Odour (Table 16): 1
Turbidity (Table 17):

Channel Characteristics

Sinuosity Type (Table 9): 1
Gradient (Table 11): 2
Entrenchment (Table 13): 1
Down's Model (Table 15): e
Sed Sorting (Table 20): Poor
Transport Mode (Table 21): 2,3
Geomorphic Units (Table 22): 5,6,9
Riffle-Pool Spacing (m): NA

Sinuosity Degree (Table 10): 1
# of Channels (Table 12): 1
Bank Failure (Table 14): 2,6
Bankfull Indicators (Table 18): 1,3,7,5
Sediment Transport Observed? ☐ Yes ☒ No ☐ Not Visible
% of Bed Active: 0
Mass Movement (Table 23): 4
% Riffles: /
% Pools: /

Bank Angle	Bank Erosion
☐ 0 - 30	☐ < 5%
☐ 30 - 60	☐ 5 - 30%
☒ 60 - 90	☐ 30 - 60%
☐ Undercut	☒ 60 - 100%

(Table 19)

	Clay/Silt	Sand	Gravel	Cobble	Boulder	Parent	Rootlets
Bank	☒	☒	☒	☒	☐	☐	☒
Riffle	☐	☐	☒	☒	☐	☐	☐
Pool	☒	☒	☒	☐	☐	☐	☐
Bed (if no riffle-pool morphology)	☐	☐	☐	☐	☐	☐	☐

Bankfull Width (m)	0.92	1.12	1.58	**Wetted Width (m)**	0.24	1.04	0.32
Bankfull Depth (m)	0.16	0.34	0.41	**Wetted Depth (m)**	0.02	0.22	0.01
Undercuts (m)	0.27	0.45	0.10	**Velocity (m/s)**	0.17	0.13	0.02
Pool Depth (m)	0.20	0.07	0.40	**Velocity Estimate Method**	WB	WB	WB
Riffle Length (m)	5.6	/	/	**Meander Amplitude (m)**	N/A	NA	NA

Notes:

- Low water levels. Small wetted perimeter
- Till exposed on bed in US extent, near reach break
- Dry gully leading from field on RB undercut. Sed. deposition observed DS of confluence
- Exposed roots + undercutting on <50% of reach. Suspend armor stone observed
- Exposed roots young and old
- Till + suspended armor stone observed on LB
- Meander amplitude not measured due to irregular/weak meanders
- Valley wall contact on left bank

Photos:

Version #4
Last edited: 04/04/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: CM

GEO MORPHIX™

Rapid Geomorphic Assessment

Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	12:45	**Reach:**	FTC-1
Weather:	Sun 25°	**Location:**	Derrycrest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Process	No.	Geomorphological Indicator	Present? Yes	Present? No	Factor Value
Evidence of Aggradation (AI)	1	Lobate bar		/	2/7
	2	Coarse materials in riffles embedded		/	
	3	Siltation in pools		/	
	4	Medial bars – Single occurance		/	
	5	Accretion on point bars		/	
	6	Poor longitudinal sorting of bed materials	/		
	7	Deposition in the overbank zone – NOT FRESH	/		
		Sum of indices =	2	5	0.29
Evidence of Degradation (DI)	1	Exposed bridge footing(s) NA	—	—	3/5
	2	Exposed sanitary / storm sewer / pipeline / etc. NA	—	—	
	3	Elevated storm sewer outfall(s) NA	—	—	
	4	Undermined gabion baskets / concrete aprons / etc. NA	—	—	
	5	Scour pools downstream of culverts / storm sewer outlets NA	—	—	
	6	Cut face on bar forms		/	
	7	Head cutting due to knickpoint migration	/		
	8	Terrace cut through older bar material		/	
	9	Suspended armour layer visible in bank	/		
	10	Channel worn into undisturbed overburden / bedrock – Near FTC-2 reach break	/		
		Sum of indices =	3	2	0.6
Evidence of Widening (WI)	1	Fallen / leaning trees / fence posts / etc.	/		4/7
	2	Occurrence of large organic debris	/		
	3	Exposed tree roots	/		
	4	Basal scour on inside meander bends		/	
	5	Basal scour on both sides of channel through riffle		/	
	6	Outflanked gabion baskets / concrete walls / etc. NA	—	—	
	7	Length of basal scour >50% through subject reach	/		
	8	Exposed length of previously buried pipe / cable / etc. NA	—	—	
	9	Fracture lines along top of bank		/	
	10	Exposed building foundation NA	—	—	
		Sum of indices =	4	3	0.57
Evidence of Planimetric Form Adjustment (PI)	1	Formation of chute(s)		/	0/7
	2	Single thread channel to multiple channel		/	
	3	Evolution of pool-riffle form to low bed relief form		/	
	4	Cut-off channel(s)		/	
	5	Formation of island(s)		/	
	6	Thalweg alignment out of phase with meander form		/	
	7	Bar forms poorly formed / reworked / removed		/	
		Sum of indices =			0

Notes:

→ Undercuts + exposed roots
→ Low flow conditions
→ Gully from farm field feeds into trib

Stability Index (SI) = (AI+DI+WI+PI)/4 = 0.37

In Regime	In Transition/Stress	In Adjustment
☐ 0.00 - 0.20	☒ 0.21 - 0.40	☐ 0.41

Version #3
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: CM Completed by: NH

GEO MORPHIX™

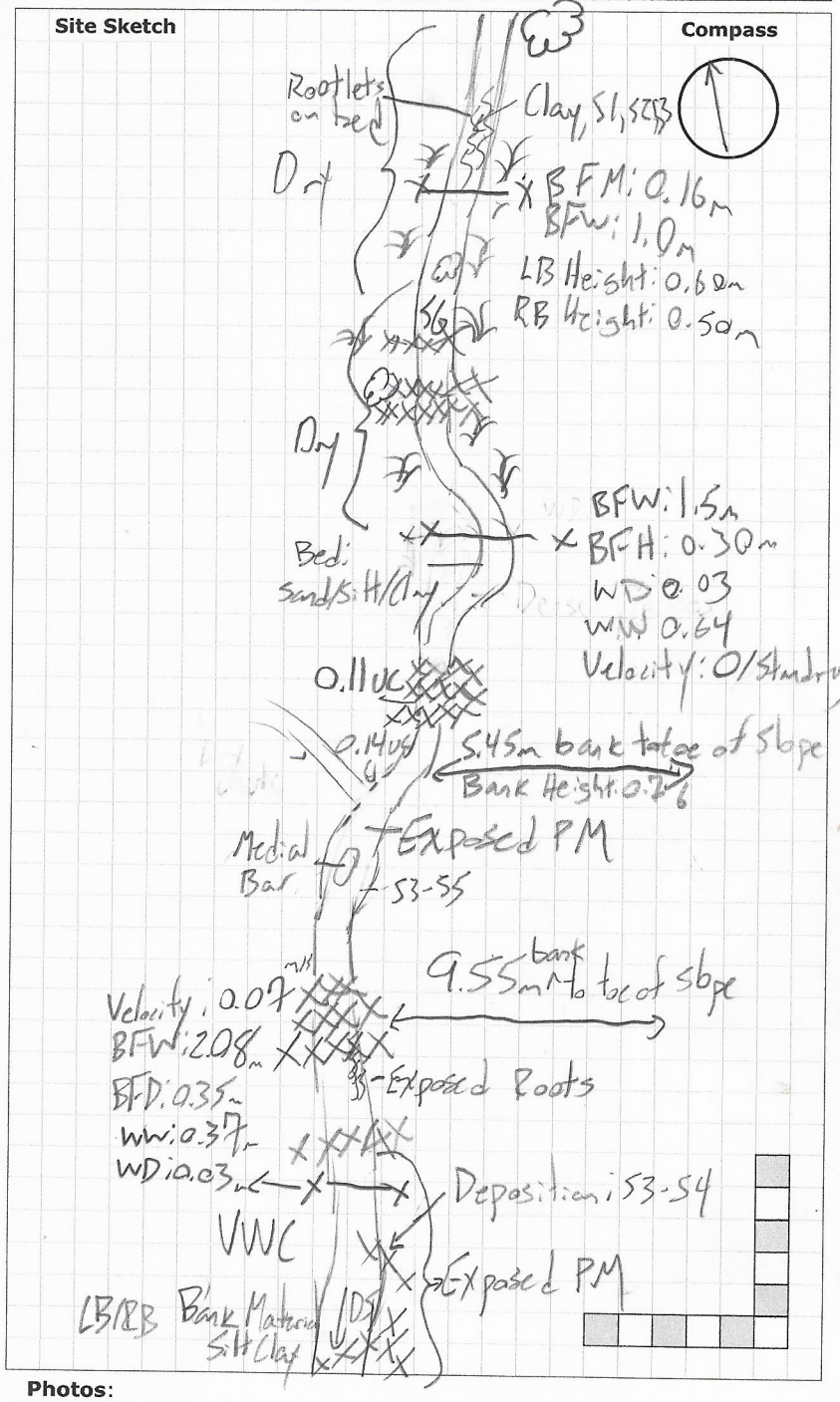
General Site Characteristics

Project Number:

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	8:09	**Reach:**	FTC-2
Weather:	24°C, Sunny	**Location:**	Derry Crest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch



Photos:

Notes:

Flow Type			
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Other			
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BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

GEO MORPHIX™

General Site Characteristics

Project Number: 25102

Date:	2025-07-15	**Stream:**	Fletcher's Creek
Time:	1:45	**Reach:**	FTC-2
Weather:	Sunny 24°C	**Location:**	Derry Crest Dr.
Field Staff:	NH, CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank		
	Bank stabilization		
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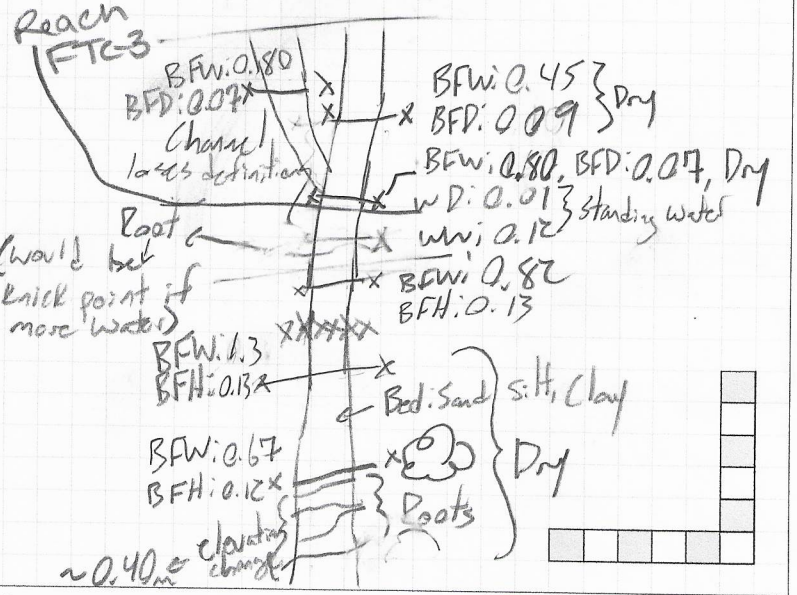
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch

Compass



Photos:

Notes:

Flow Type			
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Other			
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BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

GEO MORPHIX

Rapid Stream Assessment Technique Project Number: 25107

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	13:10	**Reach:**	FTC-2
Weather:	24°C, Sunny	**Location:**	Derry Crest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Category	Poor	Fair	Good	Excellent
Channel Stability	• < 50% of bank network stable • Recent bank sloughing, slumping or failure frequently observed	• 50-70% of bank network stable • Recent signs of bank sloughing, slumping or failure fairly common	• 71-80% of bank network stable • Infrequent signs of bank sloughing, slumping or failure	• > 80% of bank network stable • No evidence of bank sloughing, slumping or failure
	• Stream bend areas highly unstable • Outer bank height 1.2 m above stream bank (2.1 m above stream bank for large mainstem areas) • Bank overhang > 0.8-1.0 m	• Stream bend areas unstable • Outer bank height 0.9-1.2 m above stream bank (1.5-2.1 m above stream bank for large mainstem areas) • Bank overhang 0.8-0.9m	• Stream bend areas stable • Outer bank height 0.6-0.9 m above stream bank (1.2-1.5 m above stream bank for large mainstem areas) • Bank overhang 0.6-0.8 m	• Stream bend areas very stable • Height < 0.6 m above stream (< 1.2 m above stream bank for large mainstem areas) • Bank overhang < 0.6 m
	• Young exposed tree roots abundant • > 6 recent large tree falls per stream mile	• Young exposed tree roots common • 4-5 recent large tree falls per stream mile	• Exposed tree roots predominantly old and large, smaller young roots scarce • 2-3 recent large tree falls per stream mile	• Exposed tree roots old, large and woody • Generally 0-1 recent large tree falls per stream mile
	• Bottom 1/3 of bank is highly erodible material • Plant/soil matrix severely compromised	• Bottom 1/3 of bank is generally highly erodible material • Plant/soil matrix compromised	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material	• Bottom 1/3 of bank is generally highly resistant plant/soil matrix or material
	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally trapezoidally-shaped	• Channel cross-section is generally V- or U-shaped	• Channel cross-section is generally V- or U-shaped
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4 ☐ 5	☒ 6 ☐ 7 ☐ 8	☐ 9 ☐ 10 ☐ 11
Channel Scouring/ Sediment Deposition	• > 75% embedded (> 85% embedded for large mainstem areas)	• 50-75% embedded (60-85% embedded for large mainstem areas)	• 25-49% embedded (35-59% embedded for large mainstem areas)	• Riffle embeddedness < 25% sand-silt (< 35% embedded for large mainstem areas)
	• Few, if any, deep pools • Pool substrate composition >81% sand-silt	• Low to moderate number of deep pools • Pool substrate composition 60-80% sand-silt	• Moderate number of deep pools • Pool substrate composition 30-59% sand-silt	• High number of deep pools (> 61 cm deep) (> 122 cm deep for large mainstem areas) • Pool substrate composition <30% sand-silt
	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits common	• Streambed streak marks and/or "banana"-shaped sediment deposits uncommon	• Streambed streak marks and/or "banana"-shaped sediment deposits absent
	• Fresh, large sand deposits very common in channel • Moderate to heavy sand deposition along major portion of overbank area	• Fresh, large sand deposits common in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits uncommon in channel • Small localized areas of fresh sand deposits along top of low banks	• Fresh, large sand deposits rare or absent from channel • No evidence of fresh sediment deposition on overbank
	• Point bars present at most stream bends, moderate to large and unstable with high amount of fresh sand	• Point bars common, moderate to large and unstable with high amount of fresh sand	• Point bars small and stable, well-vegetated and/or armoured with little or no fresh sand	• Point bars few, small and stable, well-vegetated and/or armoured with little or no fresh sand
Point range	☐ 0 ☐ 1 ☐ 2	☐ 3 ☐ 4	☐ 5 ☒ 6	☐ 7 ☐ 8

Version #2
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: CM Completed by: NH

GEO MORPHIX™

Date:		PN:		Location:	

Category	Poor	Fair	Good	Excellent
Physical Instream Habitat	• Wetted perimeter < 40% of bottom channel width (< 45% for large mainstem areas)	• Wetted perimeter 40-60% of bottom channel width (45-65% for large mainstem areas)	• Wetted perimeter 61-85% of bottom channel width (66-90% for large mainstem areas)	• Wetted perimeter > 85% of bottom channel width (> 90% for large mainstem areas)
	• Dominated by one habitat type (usually runs) and by one velocity and depth condition (slow and shallow) (for large mainstem areas, few riffles present, runs and pools dominant, velocity and depth diversity low)	• Few pools present, riffles and runs dominant. • Velocity and depth generally slow and shallow (for large mainstem areas, runs and pools dominant, velocity and depth diversity intermediate)	• Good mix between riffles, runs and pools • Relatively diverse velocity and depth of flow	• Riffles, runs and pool habitat present • Diverse velocity and depth of flow present (i.e., slow, fast, shallow and deep water)
	• Riffle substrate composition: predominantly gravel with high amount of sand • < 5% cobble	• Riffle substrate composition: predominantly small cobble, gravel and sand • 5-24% cobble	• Riffle substrate composition: good mix of gravel, cobble, and rubble material • 25-49% cobble	• Riffle substrate composition: cobble, gravel, rubble, boulder mix with little sand • > 50% cobble
	• Riffle depth < 10 cm for large mainstem areas	• Riffle depth 10-15 cm for large mainstem areas	• Riffle depth 15-20 cm for large mainstem areas	• Riffle depth > 20 cm for large mainstem areas
	• Large pools generally < 30 cm deep (< 61 cm for large mainstem areas) and devoid of overhead cover/structure	• Large pools generally 30-46 cm deep (61-91 cm for large mainstem areas) with little or no overhead cover/structure	• Large pools generally 46-61 cm deep (91-122 cm for large mainstem areas) with some overhead cover/structure	• Large pools generally > 61 cm deep (> 122 cm for large mainstem areas) with good overhead cover/structure
	• Extensive channel alteration and/or point bar formation/enlargement	• Moderate amount of channel alteration and/or moderate increase in point bar formation/enlargement	• Slight amount of channel alteration and/or slight increase in point bar formation/enlargement	• No channel alteration or significant point bar formation/enlargement
	• Riffle/Pool ratio 0.49:1 ; ≥1.51:1	• Riffle/Pool ratio 0.5-0.69:1 ; 1.31-1.5:1	• Riffle/Pool ratio 0.7-0.89:1 ; 1.11-1.3:1	• Riffle/Pool ratio 0.9-1.1:1
	• Summer afternoon water temperature > 27°C	• Summer afternoon water temperature 24-27°C	• Summer afternoon water temperature 20-24°C	• Summer afternoon water temperature < 20°C
Point range	☐ 0 ☐ 1 ☒ 2	☐ 3 ☐ 4	☐ 5 ☐ 6	☐ 7 ☐ 8
Water Quality	• Substrate fouling level: High (> 50%)	• Substrate fouling level: Moderate (21-50%)	• Substrate fouling level: Very light (11-20%)	• Substrate fouling level: Rock underside (0-10%)
	• Brown colour • TDS: > 150 mg/L	• Grey colour • TDS: 101-150 mg/L	• Slightly grey colour • TDS: 50-100 mg/L	• Clear flow • TDS: < 50 mg/L
	• Objects visible to depth < 0.15m below surface	• Objects visible to depth 0.15-0.5m below surface	• Objects visible to depth 0.5-1.0m below surface	• Objects visible to depth > 1.0m below surface
	• Moderate to strong organic odour	• Slight to moderate organic odour	• Slight organic odour	• No odour
Point range	☐ 0 ☐ 1 ☒ 2	☐ 3 ☐ 4	☐ 5 ☐ 6	☐ 7 ☐ 8
Riparian Habitat Conditions	• Narrow riparian area of mostly non-woody vegetation	• Riparian area predominantly wooded but with major localized gaps	• Forested buffer generally > 31 m wide along major portion of both banks	• Wide (> 60 m) mature forested buffer along both banks
	• Canopy coverage: <50% shading (30% for large mainstem areas)	• Canopy coverage: 50-60% shading (30-44% for large mainstem areas)	• Canopy coverage: 60-79% shading (45-59% for large mainstem areas)	• Canopy coverage: >80% shading (> 60% for large mainstem areas)
Point range	☐ 0 ☐ 1	☐ 2 ☐ 3	☐ 4 ☒ 5	☐ 6 ☐ 7

Total overall score (0-42) = 21	Poor (<13)	Fair (13-24)	Good (25-34)	Excellent (>35)

Version #2
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: NH

GEO MORPHIX™

Reach Characteristics Project Number: 25102

Date:	2025-07-18	**Field Staff:**	NH CM	**Watershed/Subwatershed:**	
Time:	13:09	**Stream:**	Fletcher's Creek	**UTM (Upstream):**	
Weather:	Sun 25	**Reach:**	FCT-2	**UTM (Downstream):**	

Land Use (Table 1): 1 | **Valley Type** (Table 2): 3 | **Channel Type** (Table 3): 7 | **Channel Zone** (Table 4): 2 | **Flow Type** (Table 5): 2 | ☐ Evidence of Groundwater Location:_____ Photo:_____

Riparian Vegetation

Dominant Type (Table 6): 1
Encroachment (Table 7): 3

Coverage	Channel Widths	Age (yrs)
☐ None	☐ 1 - 4	☐ Immature (<5)
☐ Fragmented	☑ 4 - 10	☐ Established (5-30)
☑ Continuous	☐ > 10	☑ Mature (>30)

Aquatic & Instream Vegetation

Type (Table 8): NA
Reach Coverage %:

Woody Debris	WD Density
☑ In Cutbank	☐ Low
☑ In Channel	☐ Mod
☐ Not Present	☑ High

WDJ/50m: 2

Water Quality

Odour (Table 16): 1
Turbidity (Table 17): 2

Channel Characteristics

Sinuosity Type (Table 9): 1
Gradient (Table 11): 2
Entrenchment (Table 13): 1
Down's Model (Table 15): e
Sed Sorting (Table 20): Poor
Transport Mode (Table 21): 2,3
Geomorphic Units (Table 22): 9,10,5
Riffle-Pool Spacing (m): /

Sinuosity Degree (Table 10): 1
# of Channels (Table 12): 1
Bank Failure (Table 14): 2,6
Bankfull Indicators (Table 18): 3,5,7
Sediment Transport Observed? ☐ Yes ☐ No ☐ Not Visible
% of Bed Active: 0
Mass Movement (Table 23): 4
% Riffles: / **% Pools:** /

Bank Angle	Bank Erosion
☐ 0 – 30	☐ < 5%
☑ 30 – 60	☐ 5 – 30%
☑ 60 – 90	☑ 30 – 60%
☑ Undercut	☐ 60 – 100%

(Table 19)	Clay/Silt	Sand	Gravel	Cobble	Boulder	Parent	Rootlets
Bank	☑	☑	☑	☑	☐	☑	☐
Riffle	☐	☐	☐	☐	☐	☐	☐
Pool	☐	☐	☐	☐	☐	☐	☐
Bed (if no riffle-pool morphology)	☑	☑	☑	☑	☐	☑	☐

Bankfull Width (m)	1.0	1.50	0.67	**Wetted Width (m)**	0	0.64	0.37
Bankfull Depth (m)	0.16	0.30	0.12	**Wetted Depth (m)**	0	0.03	0.03
Undercuts (m)	0.11	0.14	/	**Velocity (m/s)**	0	0.07	0
Pool Depth (m)	/	/	/	**Velocity Estimate Method**	WB		
Riffle Length (m)				**Meander Amplitude (m)**			

Notes:

→ Left bank @ reach break w FCT-1 is ~15m from toe of slope (partially confined)
→ Dry during assessment in US extent
→ Roots growing through channel bed
→ Exposed till in bed
→ Banks become shallow + less defined moving US

Photos:

Version #4
Last edited: 04/04/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: CM

GEO MORPHIX™

Rapid Geomorphic Assessment **Project Number:** 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:		**Reach:**	FTC-2
Weather:	Sun 25°	**Location:**	Derrycrest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Process	No.	Geomorphological Indicator	Present? Yes	Present? No	Factor Value
Evidence of Aggradation (AI)	1	Lobate bar		X	1/6
	2	Coarse materials in riffles embedded		X	
	3	Siltation in pools NA			
	4	Medial bars - one occurance DS of confluence		X	
	5	Accretion on point bars		X	
	6	Poor longitudinal sorting of bed materials	X		
	7	Deposition in the overbank zone		X	
		Sum of indices =	1	5	0.17
Evidence of Degradation (DI)	1	Exposed bridge footing(s)	NA		2/5
	2	Exposed sanitary / storm sewer / pipeline / etc.	NA		
	3	Elevated storm sewer outfall(s)	NA		
	4	Undermined gabion baskets / concrete aprons / etc.	NA		
	5	Scour pools downstream of culverts / storm sewer outlets	NA		
	6	Cut face on bar forms		/	
	7	Head cutting due to knickpoint migration		/	
	8	Terrace cut through older bar material		/	
	9	Suspended armour layer visible in bank	✓		
	10	Channel worn into undisturbed overburden / bedrock	✓		
		Sum of indices =	2	3	0.4
Evidence of Widening (WI)	1	Fallen / leaning trees / fence posts / etc.	/		3/7
	2	Occurrence of large organic debris	/		
	3	Exposed tree roots	/		
	4	Basal scour on inside meander bends		/	
	5	Basal scour on both sides of channel through riffle		/	
	6	Outflanked gabion baskets / concrete walls / etc.	NA		
	7	Length of basal scour >50% through subject reach		/	
	8	Exposed length of previously buried pipe / cable / etc.	NA		
	9	Fracture lines along top of bank		/	
	10	Exposed building foundation	NA		
		Sum of indices =	3	4	0.43
Evidence of Planimetric Form Adjustment (PI)	1	Formation of chute(s)		/	0/7
	2	Single thread channel to multiple channel		/	
	3	Evolution of pool-riffle form to low bed relief form		/	
	4	Cut-off channel(s)		/	
	5	Formation of island(s)		/	
	6	Thalweg alignment out of phase with meander form		/	
	7	Bar forms poorly formed / reworked / removed		/	
		Sum of indices =	0	7	0

Notes:

Stability Index (SI) = (AI+DI+WI+PI)/4 = 0.21

In Regime	In Transition/Stress	In Adjustment
☐ 0.00 - 0.20	☒ 0.21 - 0.40	☐ 0.41

Version #3
Last edited: 10/02/2023

Senior staff sign-off (if required): _____ Checked by: CM Completed by: NH

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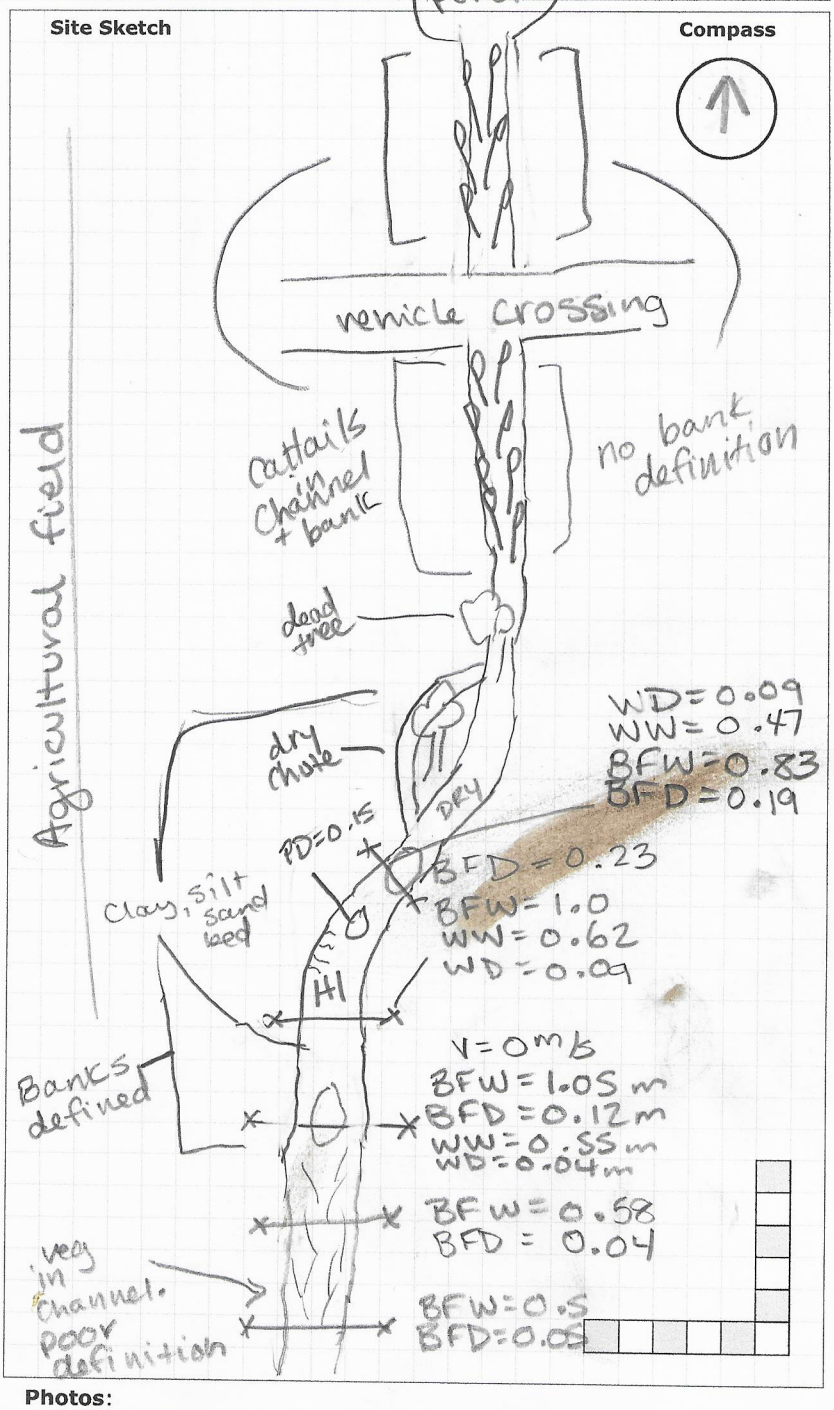
General Site Characteristics

Project Number: 25102

Date:	2025-07-18	**Stream:**	Fletcher's Creek
Time:	14:15	**Reach:**	FTC-3
Weather:	24°C Sunny	**Location:**	Derry Crest Dr.
Field Staff:	NH CM	**Watershed/Subwatershed:**	

Features		Monitoring	
	Reach break		Long-profile
	Station location		Monumented XS
	Cross-section		Monumented photo
	Flow direction		Monumented photo direction
	Riffle		Sediment sampling
	Pool		Erosion pins
	Sediment bar		Scour chains
	Eroded bank/slope	**Additional Symbols**	
	Undercut bank	cattails/phrag	
	Bank stabilization		
	Leaning tree		
	Fence		
	Culvert/outfall		
	Swamp/wetland		
	Grasses		
	Tree		
	Instream log/tree		
	Woody debris		
	Beaver dam		
	Vegetated island		

Site Sketch



Flow Type

H1	Standing water	**H1A**	Back water
H2	Scarcely perceptible flow		
H3	Smooth surface flow		
H4	Upwelling		
H5	Rippled		
H6	Unbroken standing wave		
H7	Broken standing wave		
H8	Chute		
H9	Free fall	**H9A**	Dissipates below free fall

Substrate

S1	Silt	**S6**	Small boulder
S2	Sand	**S7**	Large boulder
S3	Gravel	**S8**	Bimodal
S4	Small cobble	**S9**	Bedrock/till
S5	Large cobble		

Other

BM	Benchmark	**EP**	Erosion pin
BS	Backsight	**RB**	Rebar
DS	Downstream	**US**	Upstream
WDJ	Woody debris jam	**TR**	Terrace
VWC	Valley wall contact	**FC**	Flood chute
BOS	Bottom of slope	**FP**	Flood plain
TOS	Top of slope	**KP**	Knick point

Photos:

Notes:

→ Small extent of reach has defined banks, but is otherwise grassy swale (DS) or cattail/phrag encroached wetland (US)

Version #4
Last edited: 21/02/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: CM

GEO MORPHIX™

Reach Characteristics Project Number: 25102

Date:	2025-07-18	**Field Staff:**	CM NH	**Watershed/Subwatershed:**	
Time:	14:15	**Stream:**	Fletcher's Creek	**UTM (Upstream):**	
Weather:	24° Sunny	**Reach:**	FCT-3	**UTM (Downstream):**	

Land Use (Table 1): 3,9 | **Valley Type** (Table 2): 1 | **Channel Type** (Table 3): 12 | **Channel Zone** (Table 4): 2 | **Flow Type** (Table 5): 2 | ☐ Evidence of Groundwater Location:_____ Photo:_____

Riparian Vegetation

Dominant Type (Table 6): 3
Encroachment (Table 7): 90

Coverage	Channel Widths	Age (yrs)
☐ None	☐ 1 - 4	☒ Immature (<5)
☐ Fragmented	☒ 4 - 10	☐ Established (5-30)
☒ Continuous	☐ > 10	☐ Mature (>30)

Aquatic & Instream Vegetation

Type (Table 8): 1
Reach Coverage %: 60

Woody Debris	WD Density
☒ In Cutbank	☒ Low
☐ In Channel	☐ Mod
☐ Not Present	☐ High

WDJ/50m: 0.25

Water Quality

Odour (Table 16): 1
Turbidity (Table 17): 3

Channel Characteristics

Sinuosity Type (Table 9): 1
Gradient (Table 11): 1
Entrenchment (Table 13): 4
Down's Model (Table 15): 5
Sed Sorting (Table 20): well
Transport Mode (Table 21): 3
Geomorphic Units (Table 22): 5
Riffle-Pool Spacing (m): /

Sinuosity Degree (Table 10): 1
# of Channels (Table 12): 1
Bank Failure (Table 14): 1
Bankfull Indicators (Table 18): 1,3,5
Sediment Transport Observed? ☐ Yes ☒ No ☐ Not Visible
% of Bed Active: 0
Mass Movement (Table 23): /
% Riffles: 0
% Pools: 15

Bank Angle	Bank Erosion
☐ 0 - 30	☐ < 5%
☒ 30 - 60	☒ 5 - 30%
☐ 60 - 90	☐ 30 - 60%
☐ Undercut	☐ 60 - 100%

(Table 19)

	Clay/Silt	Sand	Gravel	Cobble	Boulder	Parent	Rootlets
Bank	☒	☒	☒	☐	☐	☐	☐
Riffle	☐	☐	☐	☐	☐	☐	☐
Pool	☐	☐	☐	☐	☐	☐	☐
Bed (if no riffle-pool morphology)	☒	☒	☒	☐	☐	☐	☐

Bankfull Width (m)	0.5	1.05	0.83	**Wetted Width (m)**	0.55	0.62	0.47
Bankfull Depth (m)	0.05	0.12	0.19	**Wetted Depth (m)**	0	0	0.04
Undercuts (m)	/	/	/	**Velocity (m/s)**	0	0	0
Pool Depth (m)	0.15	0.09	0.04	**Velocity Estimate Method**	WB →		
Riffle Length (m)	/	/	/	**Meander Amplitude (m)**	/	/	/

Notes:

→ Sporadic stagnant puddles but predominantly dry channel
→ Banks have little to no definition DS at FCT-2 reach break or in US extent that is dominated by cattails and phragmites
→ Small section of reach that was not swale/wetland had exposed roots
→ 1 dry chute present

Photos:

Version #4
Last edited: 04/04/2023

Senior staff sign-off (if required): _____ Checked by: _____ Completed by: CM

Appendix E:
Detailed Assessment Summaries

Detailed Geomorphological Assessment Summary

Reach FCT-1

Project Number:	PN 18129	**Date:**	Nov 1, 2018
Client:	DeZen Realty	**Length Surveyed (m):**	82.6
Location:	Hurontario Street and Derry Road West	**# of Cross-Sections:**	8

Reach Characteristics

Drainage Area:	0.672 km^2	**Dominant Riparian Vegetation Type:**	Herbaceous, trees
Geology/Soils:	Till	**Extent of Riparian Cover:**	Continuous
Surrounding Land Use:	Agricultural, commerical	**Width of Riparian Cover:**	4 to 10 times channel width
Valley Type:	Confined	**Age Class of Riparian Vegetation:**	Established (5-10 yrs)
Dominant Instream Vegetation Type:	None	**Extent of Encroachment into Channel:**	Heavy
Portion of Reach with Vegetation:	0	**Density of Woody Debris:**	High

Hydrology

Measured Discharge (m^3/s):	0.01	**Calculated Bankfull Discharge (m^3/s):**	3.27
Modelled 2-year Discharge (m^3/s):	Not modelled	**Calculated Bankfull Velocity (m/s):**	2.89
Modelled 2-year Velocity (m/s):	Not modelled		

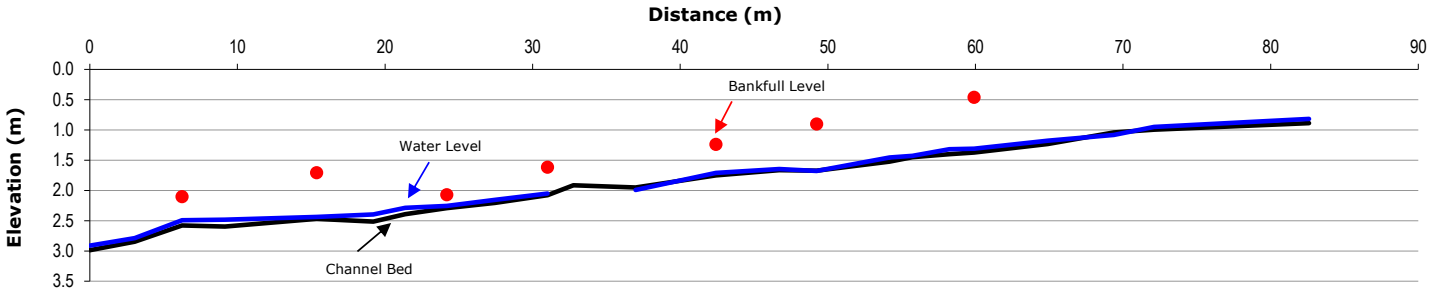
Profile Characteristics

Bankfull Gradient (%):	3.33
Channel Bed Gradient (%):	2.55
Riffle Gradient (%):	Not measured
Riffle Length (m):	Not measured
Riffle-Pool Spacing (m):	Not measured

Planform Characteristics

Sinuosity:	1.10
Meander Belt Width (m):	Not measured
Radius of Curvature (m):	Not measured
Meander Amplitude (m):	Not measured
Meander wavelength (m):	Not measured

Longitudinal Profile



Bank Characteristics

	Minimum	Maximum	Average
Bank Height (m):	0.2	1.30	0.91
Bank Angle (deg):	25	90	72
Root Depth (m):	0.00	0.45	0.22
Root Density (%):	0	10	6
Bank Undercut (m):	0	0.4	0.10

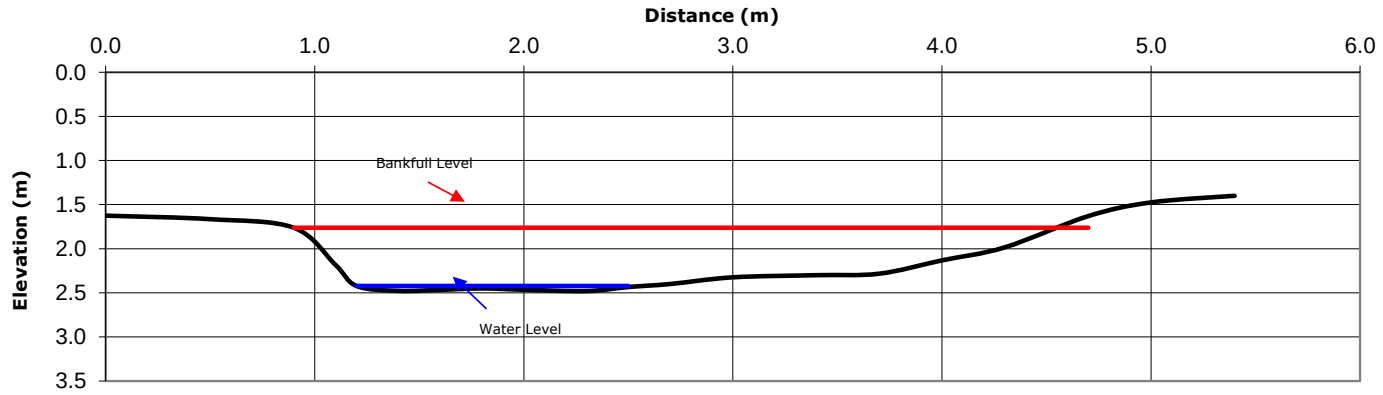
	Minimum	Maximum	Average
Torvane Value (kg/cm^2):		Not measured	
Penetrometer Value (kg/cm^3):		Not measured	
Bank Material (range):		Clay to silt	

Cross-Sectional Characteristics

	Minimum	Maximum	Average
Bankfull Width (m):	1.30	3.40	2.25
Average Bankfull Depth (m):	0.16	0.90	0.50
Bankfull Width/Depth (m/m):	2	8	5
Wetted Width (m):	0.55	1.30	0.93
Average Water Depth (m):	0.00	0.08	0.05
Wetted Width/Depth (m/m):	8	68	26
Entrenchment (m):		Not measured	
Entrenchment Ratio (m/m):		Not measured	
Maximum Water Depth (m):	0.03	0.14	0.07
Manning's *n* :		0.040	

Photograph at cross section 4 (looking downstream)

Representative Cross-Section 2



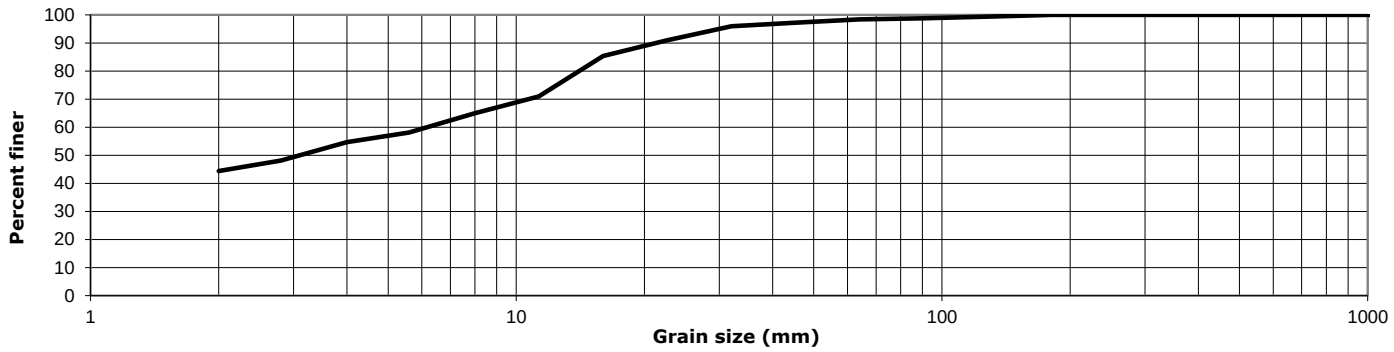
Substrate Characteristics

Particle Size (mm)

D_{10} :	2.0
D_{50} :	3.2
D_{84} :	15.0

Subpavement:	Till
Particle shape:	Platy
Embeddedness (%):	10 to 50
Particle range (riffle):	Clay to cobble
Particle Range (pool):	Clay to gravel

Cumulative Particle Size Distribution



Channel Thresholds

Flow Competency (m/s):		**Tractive Force at Bankfull (N/m^2):**	164.37
for D_{50}:	0.34	**Tractive Force at 2-year flow (N/m^2):**	Not modelled
for D_{84}:	0.69	**Critical Shear Stress (D_{50}) (N/m^2):**	2.33
Unit Stream Power at Bankfull (W/m^2):	474.20		

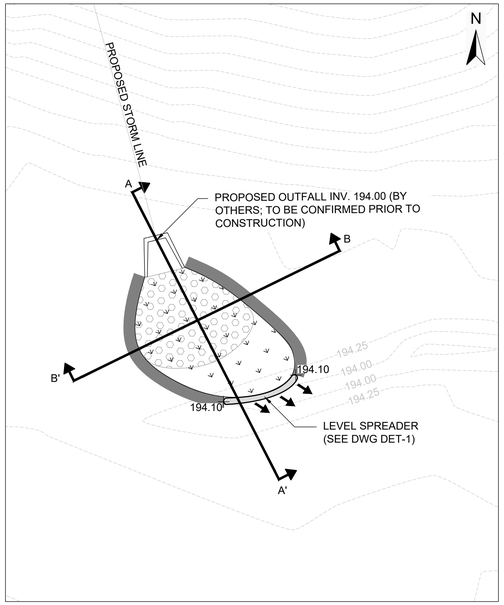
General Field Observations

Channel Description

Reach FCT-1 was a mixed-load meandering channel, with a moderate gradient that sits within a confined valley. The channel flowed through a continuous riparian buffer zone that extended 4 to 10 times the channel width and contained predominately trees and herbaceous vegetation. The channel was heavily encroached by vegetation. High density of woody debris was observed in the channel. The average bankfull width and depth were 2.44 m and 0.50 m. The average wetted width and depth were 0.93 m and 0.05 m. Bank angles ranged from 25 to 90 degrees, with undercutting measured up to 0.4 m. Bed materials ranged from clay to cobbles in riffles, and clay to gravel in pools. Bank material consisted of clay and silt.

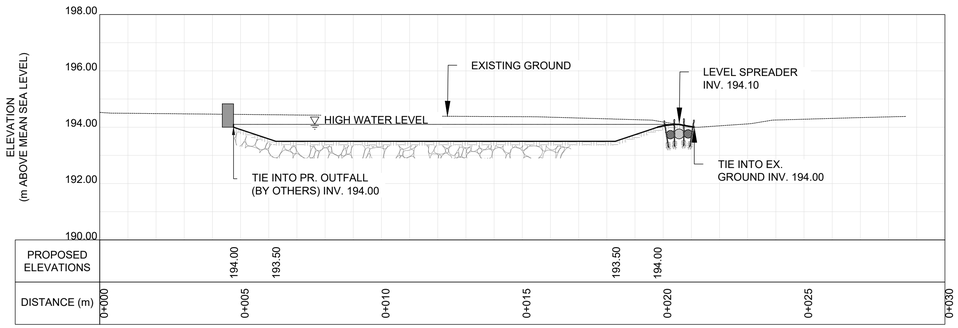
Cross Section 6 - Facing Upstream

Appendix F:
Outfall Design Drawings

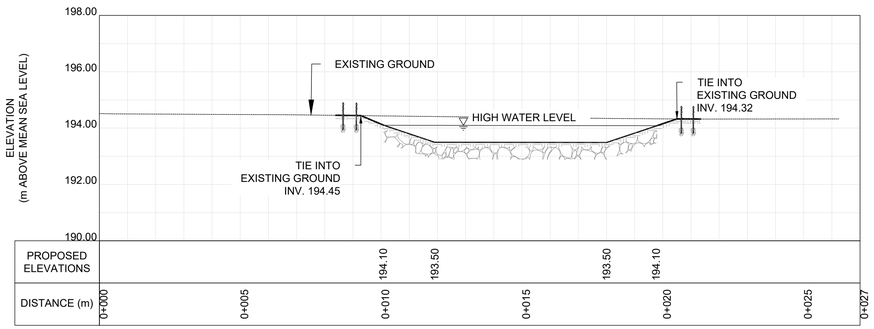


PLANFORM
1:250

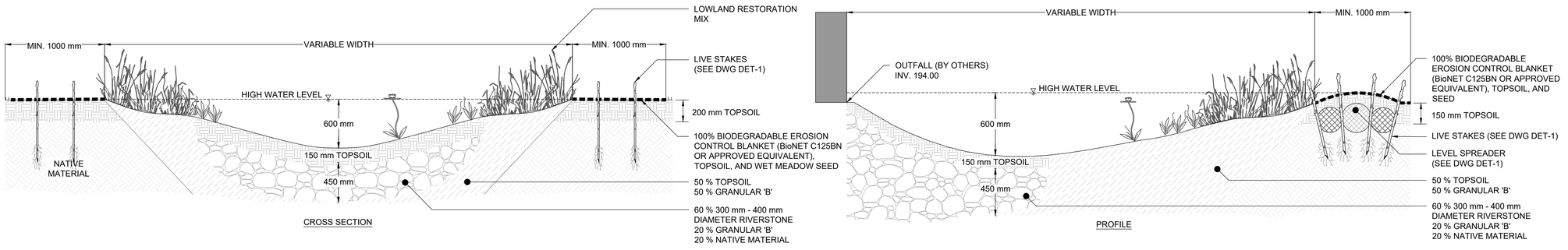
LEGEND
- SCONED (SEE DWG GEO-1)
- SCONED (SEE DWG GEO-1)
- 100% BIODEGRADABLE EROSION CONTROL BLANKET AND LIVE STAKES (SEE DWG DET-1)
- LEVEL SPREADER (SEE DWG DET-1)



CROSS SECTION A-A'
1:100

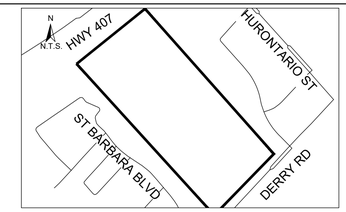


CROSS SECTION B-B'
1:100



STONE CORE NATURALIZED ENERGY DISSIPATER
N.T.S

LEGEND
- TOPSOIL
- STONE CORE (SEE DWG GEO-1)
- 100% BIODEGRADABLE EROSION CONTROL BLANKET, WET MEADOW SEED AND LIVE STAKES (SEE DWG DET-1)
- LEVEL SPREADER (SEE DWG DET-1)



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1.0	08/15/2025	AS	FIRST DETAILED DESIGN SUBMISSION TO CLIENT
	DATE	BY	REVISIONS

DESIGNED BY: AS | CHECKED BY: PV

DRAWN BY: RS | DATE: AUGUST 2025

DRAFT FOR INTERNAL DISCUSSION

NOT FOR CONSTRUCTION

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Campbellville, Ontario L0P 1B0
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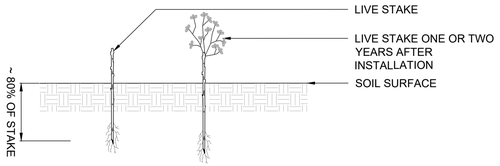
DEZEN REALTY DEVELOPMENT
7140 HURONTARIO STREET
CITY OF MISSISSAUGA

FLETCHERS CREEK
SWM OUTFALL DESIGN SUPPORT
PLANFORM AND PROFILE

PROJECT No.: PN25102 | DRAWING No.: GEO-1

SCALE: AS NOTED | SHEET 1 OF 2

SCALED FOR PLOT ON 'ARCH D'



SCIENTIFIC NAME	COMMON NAME	QTY	CONDITION
CORNUS STOLONIFERA	RED OSIER DOGWOOD	34	1 m, LIVE STAKE
SALIX BEBBIANA	BEBB'S WILLOW	34	1 m, LIVE STAKE
SALIX DISCOLOR	PUSSY WILLOW	34	1 m, LIVE STAKE
SALIX INTERIOR	SANDBAR WILLOW	34	1 m, LIVE STAKE
SALIX LUCIDA	SHINING WILLOW	34	1 m, LIVE STAKE

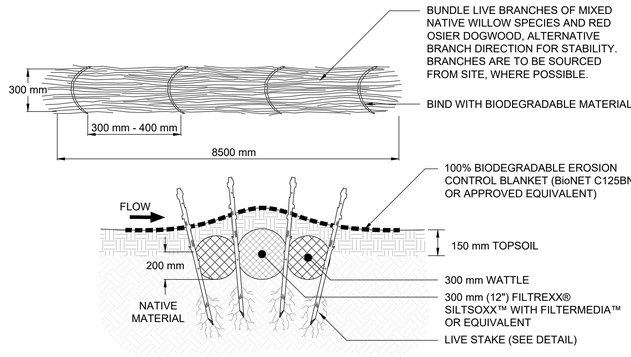
NOTES

1. QUANTITY TO BE DETERMINED BASED ON AREA OF DISTURBANCE TO BE RESTORED
2. LIVE STAKES SHOULD BE FROM AT MINIMUM 2-YEAR OLD STOCK.
3. LIVE STAKES ARE TO BE INSTALLED AT A DENSITY OF 3 STAKES PER SQUARE METRE.
4. LIVE STAKES SHOULD BE PRE-SOAKED (SUBMERGED IN WATER) FOR AT LEAST 24 HOURS AFTER HARVESTING AND IMMEDIATELY BEFORE INSTALLATION.
5. LIVE STAKES SHOULD NOT BE STORED FOR A PERIOD LONGER THAN 2 DAYS, UNLESS THEY ARE BEING SOAKED.
6. THE CONTRACTOR SHALL PROTECT PLANT MATERIALS FROM DRYING FROM THE TIME OF HARVEST UNTIL INSTALLED.
7. LIVE STAKES ARE TO BE A MINIMUM OF 25 mm IN DIAMETER AND CUT TO A LENGTH OF 1000 mm.
8. CUT ANGLE AT THE BOTTOM OF THE STAKE AND FLAT ON THE TOP.
9. TRIM ALL SIDE BRANCHES WHILE TAKING CARE NOT TO DAMAGE THE BARK.
10. INSTALL STAKES WITH BUDS POINTING UPWARDS AND THICKER STEM IN THE BED.
11. LIVE STAKES SHOULD BE INSTALLED USING A LARGE RUBBER MALLET.
14. IN COMPACT SOIL A PILOT HOLE MUST BE USED TO LIMIT DAMAGE TO THE STAKES. PILOT HOLES SHOULD BE MAX. 25 mm DIAMETER.
15. IF USING A PILOT HOLE REPACK SOIL AROUND THE LIVE STAKE.
16. 80% OF THE STAKE IS TO BE BELOW SURFACE.
17. TAMP THE LIVE STAKE INTO THE GROUND AT RIGHT ANGLE TO THE SURFACE.
18. LIVE STAKES SHOULD STAND FIRM FROM THE SOIL FOLLOWING INSTALLATION.
19. ALL STAKES NOT PLANTED TO THE SPECIFICATIONS ABOVE WILL BE REPLACED AT THE CONTRACTOR'S EXPENSE.

LIVE STAKING
N.T.S.

EROSION CONTROL BLANKET SPECIFICATIONS

1. A BIODEGRADABLE EROSION CONTROL BLANKET (ECB) SHALL BE INSTALLED ON ALL DISTURBED/NATURAL SURFACES FOLLOWING THE PLACEMENT OF TOPSOIL AND APPLICATION OF THE NATIVE SEED MIX.
2. THE ECB MUST BE CONSTRUCTED OF 100% WOVEN COCONUT FIBRE (E.G., COIR) OR STRAW MAT WITHIN A GEOJUTE NETTING (TOP AND BOTTOM) WITH BIODEGRADABLE THREAD. NON-BIODEGRADABLE MATERIAL INCLUDING POLYPROPELENE OR PLASTICS WITH A BIODEGRADABLE RATING ARE NOT ACCEPTABLE. THE MINIMUM WEIGHT OF THE ECB MUST BE 400 g/m^2 (12 oz./yd^2).
3. TO INSTALL, THE ECB MUST BE UNROLLED DOWNSLOPE OR IN DIRECTION OF WATER FLOW. ADJACENT ECBS SHOULD OVERLAP A MINIMUM OF 150 mm ALONG THE EDGES. AT THE END OF EACH ROLL, FOLD BACK 100 mm TO 200 mm OF THE ECB. OVERLAP THIS 100 mm TO 200 mm OVER THE START OF THE NEXT ROLL. SECURE THE TWO LAYERS TO THE GROUND SECURELY.
4. BIODEGRADABLE OR TAPERED WOODEN STAKES SHALL BE USED TO SECURE THE BLANKET. STAKES SHALL BE INSTALLED AT THE SPACING RECOMMENDED BY THE ECB MANUFACTURER TO PREVENT SURFACE RUNOFF FROM ERODING THE UNDERLYING SOIL.



NOTES

1. WATTLE AND SILTSOXX TO BE INSTALLED IN A 200 mm WIDE TRENCH AND STAKED EVERY METER.
2. WATTLE, SILTSOXX AND GRADING INSTALLATION TO RESULT IN A LEVEL BERM WITHOUT BREAKS.

WATTLE AND FILTREXX® SILTSOXX™ (OR EQUIVALENT) DETAIL
N.T.S.

CVC 3 – LOWLAND RESTORATION MIX

SCIENTIFIC NAME	COMMON NAME	PERCENTAGE
ANEMONE CANADENSIS	CANADA ANEMONE	1
BIDENS CERNUA	NODDING BEAGGARTICKS	1
CAREX VULPINOIDEA	FOX SEDGE	25
ELYMUS VIRGINICUS VAR. VIRGINICUS	VIRGINIA WILD RYE	25
EUTROCHIUM MACULATUM VAR. MACULATUM	SPOTTED JOE PYE WEED	1
JUNCUS EFFUSUS SUBSP. SOLUTUS	EASTERN SOFT RUSH	5
JUNCUS TENUIS	PATH RUSH	5
POA PALUSTRIS	FOWL BLUEGRASS	25
SCIRPUS ATROVIRENS	DARK GREEN BULRUSH	5
SYMPHYOTRICHUM NOVAE-ANGLIAE	NEW ENGLAND ASTER	1
SYMPHYOTRICHUM PUNICEUM	SWAMP ASTER	1
VERBENA HASTATA	BLUE VERVAIN	5

NOTES:

1. APPLY SEED MIX AT A RATE OF 25 kg PER HECTARE.
2. SEEDING SHALL OVERLAP ADJACENT GROUND COVER BY 300 mm.
3. SIMULTANEOUSLY APPLY THE SPECIED NURSE CROP MIX AT A RATE OF 15 kg PER HECTARE.
4. WATER SOIL AFTER SEED APPLICATION.

CVC RESTORATION NURSE CROP MIX

SCIENTIFIC NAME	COMMON NAME	PERCENTAGE
AVENA SATIVA	ANNUAL OATS	40
ELYMUS CANADENSIS	CANADA WILD RYE	15
HORDEUM VULGARE	BARLEY	45

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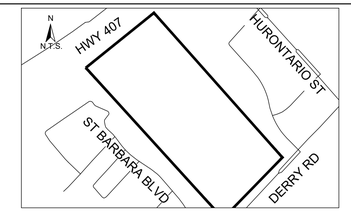
4. WATER SOIL AFTER SEED APPLICATION.
5. IF SEEDING IN FALL (OCTOBER-NOVEMBER), 100% WINTER WHEAT (TRITICUM AESTIVUM) SHOULD BE SUBSTITUTED FOR THE SPECIES LISTED ABOVE.

SCALED FOR PLOT ON 'ARCH D'



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DEZEN REALTY DEVELOPMENT
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FLETCHERS CREEK SWM OUTFALL DESIGN SUPPORT PLANFORM AND PROFILE

PROJECT No.: PN25102	DRAWING No.: DET-1
SCALE: AS NOTED	SHEET 2 OF 2

APPENDIX C

CULTEC SYSTEM – EROSION CONTROL VOLUME CALCULATIONS
ORIFICE SIZING & SPLITTER MANHOLE DETAILS
QUALITY CONTROL OIL/GRIT SEPARATOR CALCULATIONS

SKIRA & ASSOCIATES LTD.
CONSULTING ENGINEERS

WORKSHEET
for Circular Orifice -Erosion control

Project Description	
Worksheet	Orifice - tube
Type	Circular Orifice
Solve For	Diameter

Input Data		
Discharge	0.0180	m^3/s
Headwater Elevation	197.50	m
Centroid Elevation	195.42	m
Tailwater Elevation	195.35	m
Discharge Coefficient	0.60	

Results		
Diameter	77	mm
Headwater Heighy Above	2.08	m
Tailwater Height Above	-0.07	m
Flow Area	4.69E-03	m^2
Velocity	6.39	m/s

SKIRA & ASSOCIATES LTD.
CONSULTING ENGINEERS

WORKSHEET
for Circular Orifice -100 yr

Project Description	
Worksheet	Orifice - tube
Type	Circular Orifice
Solve For	Diameter

Input Data		
Discharge	0.323	m^3/s
Headwater Elevation	201.20	m
Centroid Elevation	194.72	m
Tailwater Elevation	194.60	m
Discharge Coefficient	0.60	

Results		
Diameter	247	mm
Headwater Heighy Above	6.48	m
Tailwater Height Above	-0.12	m
Flow Area	4.77E-02	m^2
Velocity	11.28	m/s

Hydroworks Sizing Summary

Dezen Vicksburg - Phase 2

12-20-2024

Recommended Size: HydroDome HD 8

Hydroworks Sizing Program Version 5.8.5

A HydroDome HD 8 is recommended to provide 80 % annual TSS removal based on a drainage area of 7.77 (ha) with an imperviousness of 80 % and Toronto Bloor St., Ontario rainfall for the Hydroworks standard particle size distribution.

The recommended HydroDome HD 8 treats 100 % of the annual runoff and provides 83 % annual TSS removal for the Toronto Bloor St. rainfall records and Hydroworks standard particle size distribution.

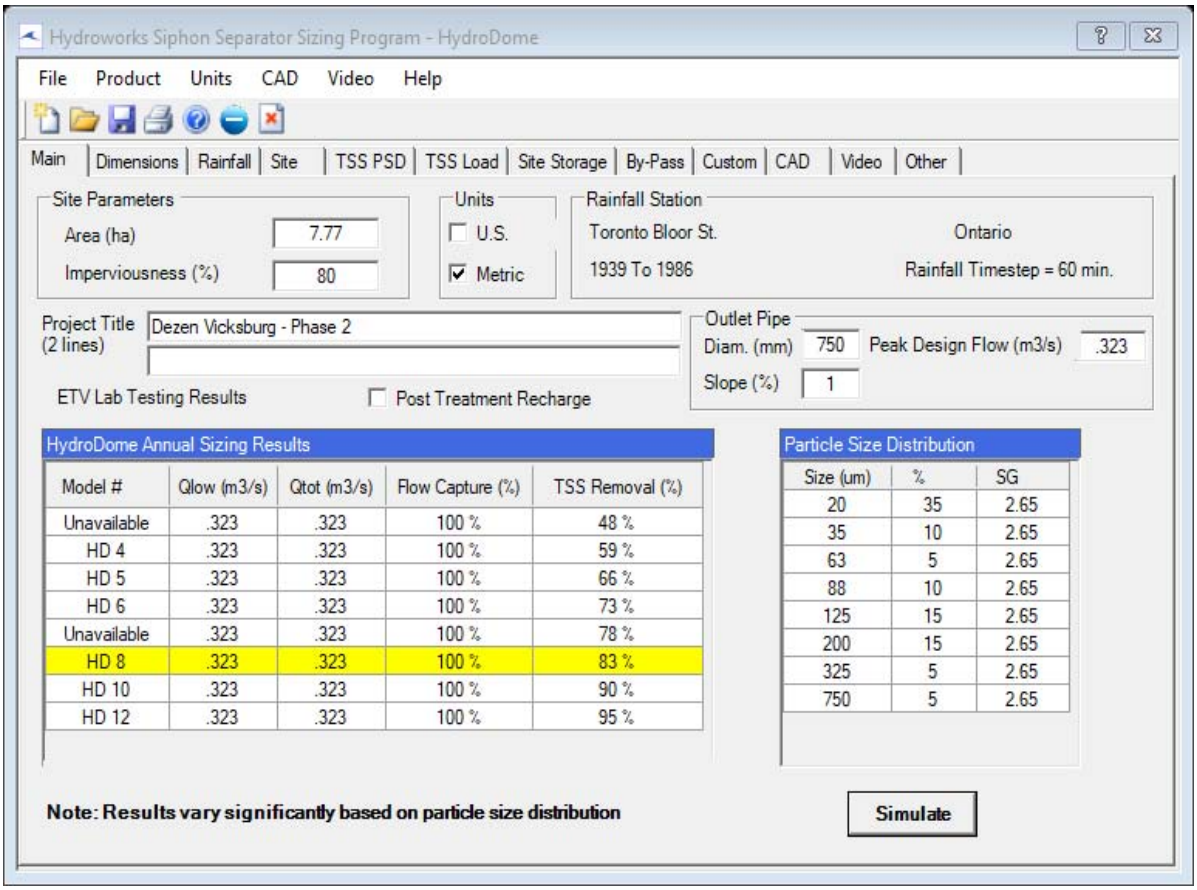
The HydroDome has a siphon which creates a discontinuity in headloss. The given peak flow of .323 (m3/s) Is less than the full pipe flow of 1.11 (m3/s) indicating free flow in the pipe during the peak flow assuming no tailwater condition. Partial pipe flow was assumed for the headloss calculations. The headloss was calculated to be 377 (mm) above the crown of the 750 (mm) outlet pipe.

This summary report provides the main parameters that were used for sizing. These parameters are shown on the summary tables and graphs provided in this report.

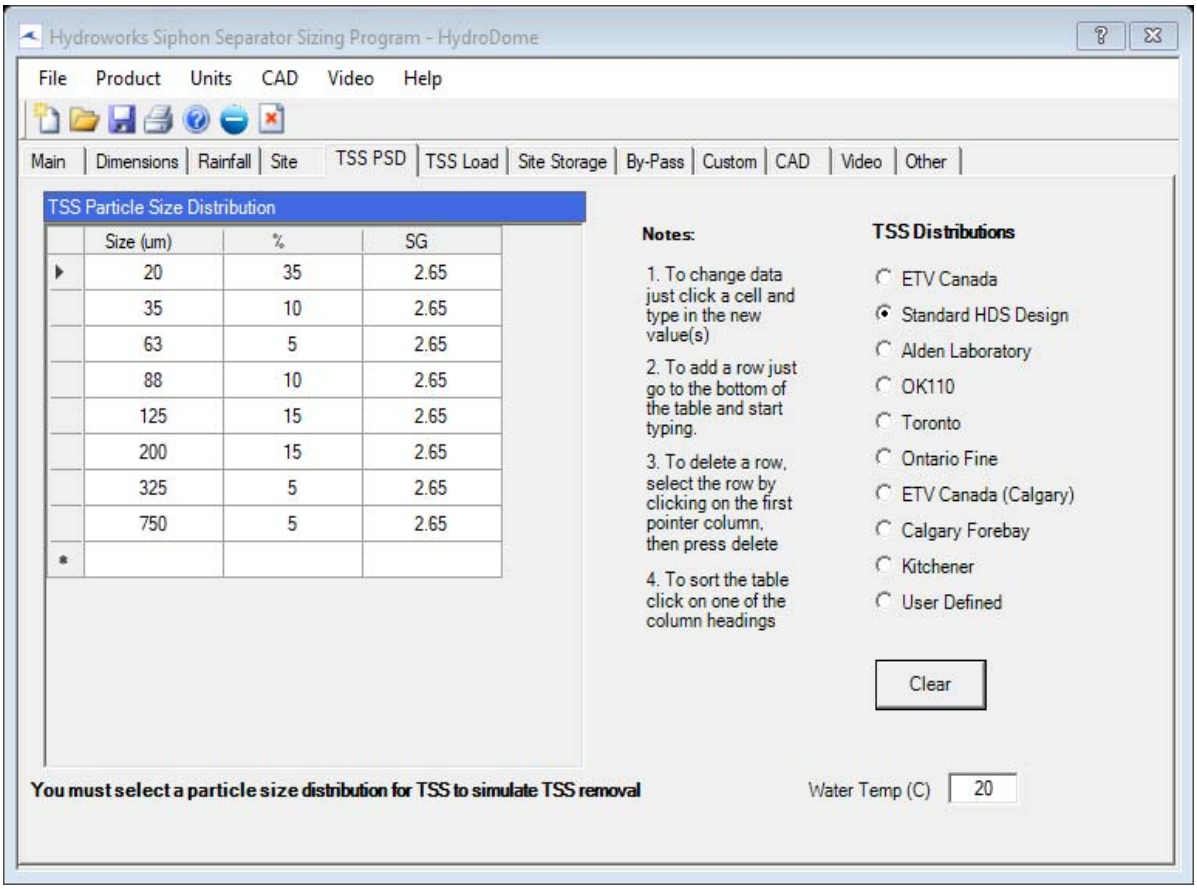
If you have any questions regarding this sizing summary please do not hesitate to contact Hydroworks at 888-290-7900 or email us at support@hydroworks.com.

The sizing program is for sizing purposes only and does not address any site specific parameters such as hydraulic gradeline, tailwater submergence, groundwater, soils bearing capacity, etc. Headloss calculations are not a hydraulic gradeline calculation since this requires a starting water level and an analysis of the entire system downstream of the HydroDome .

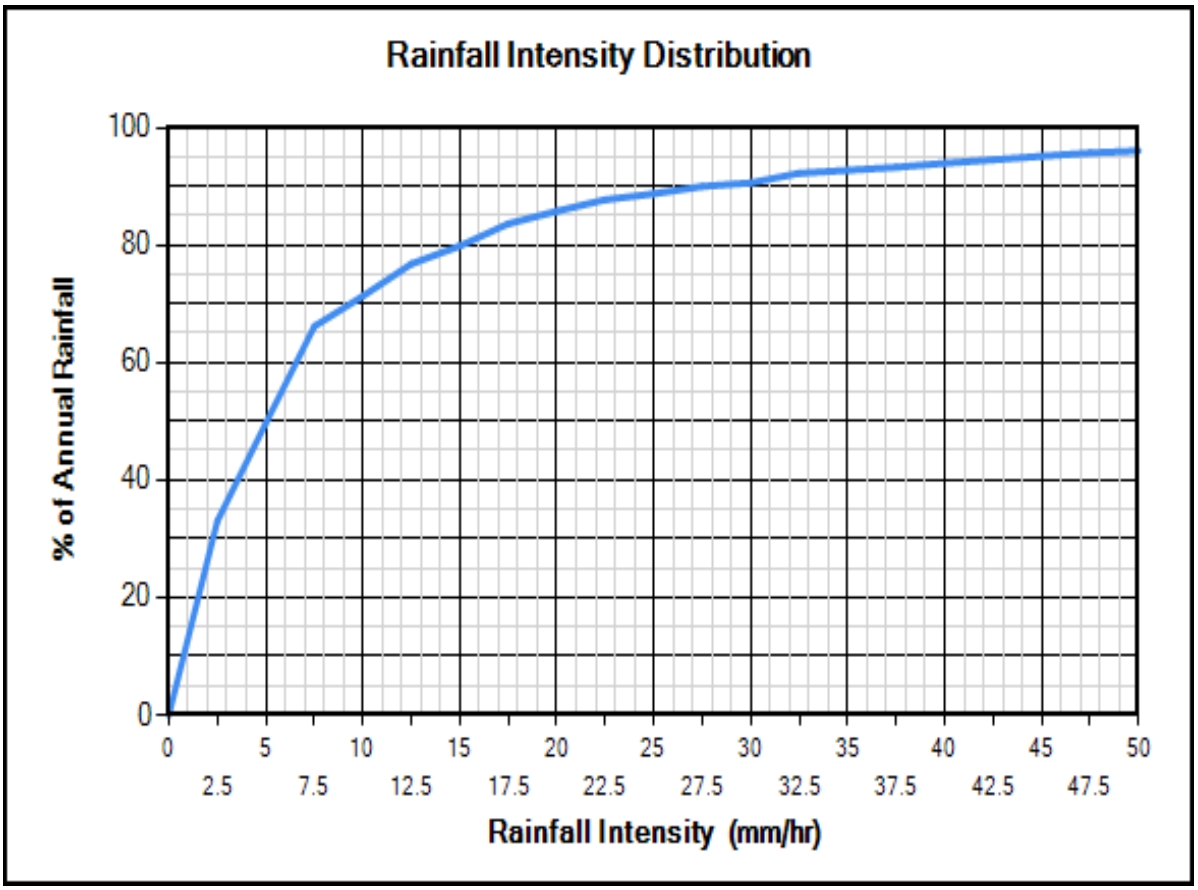
TSS Removal Sizing Summary



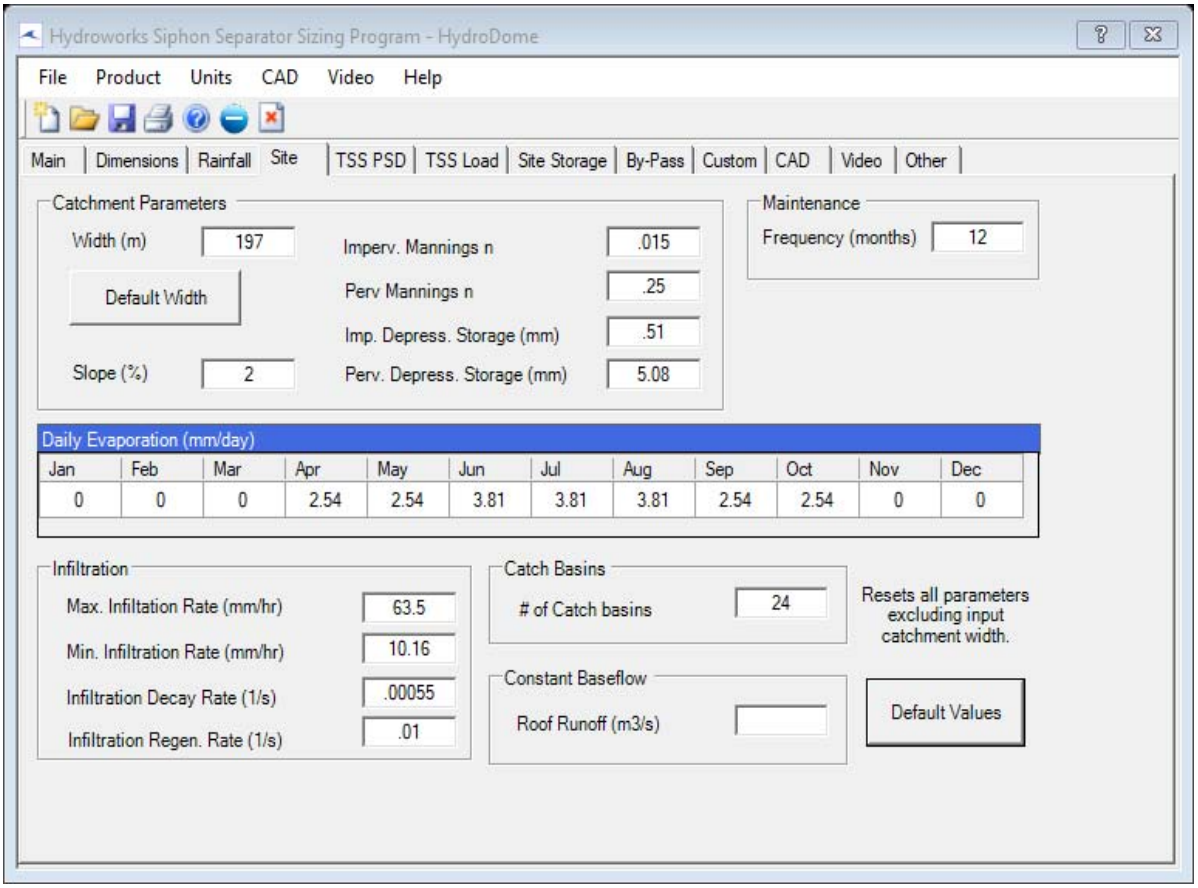
TSS Particle Size Distribution



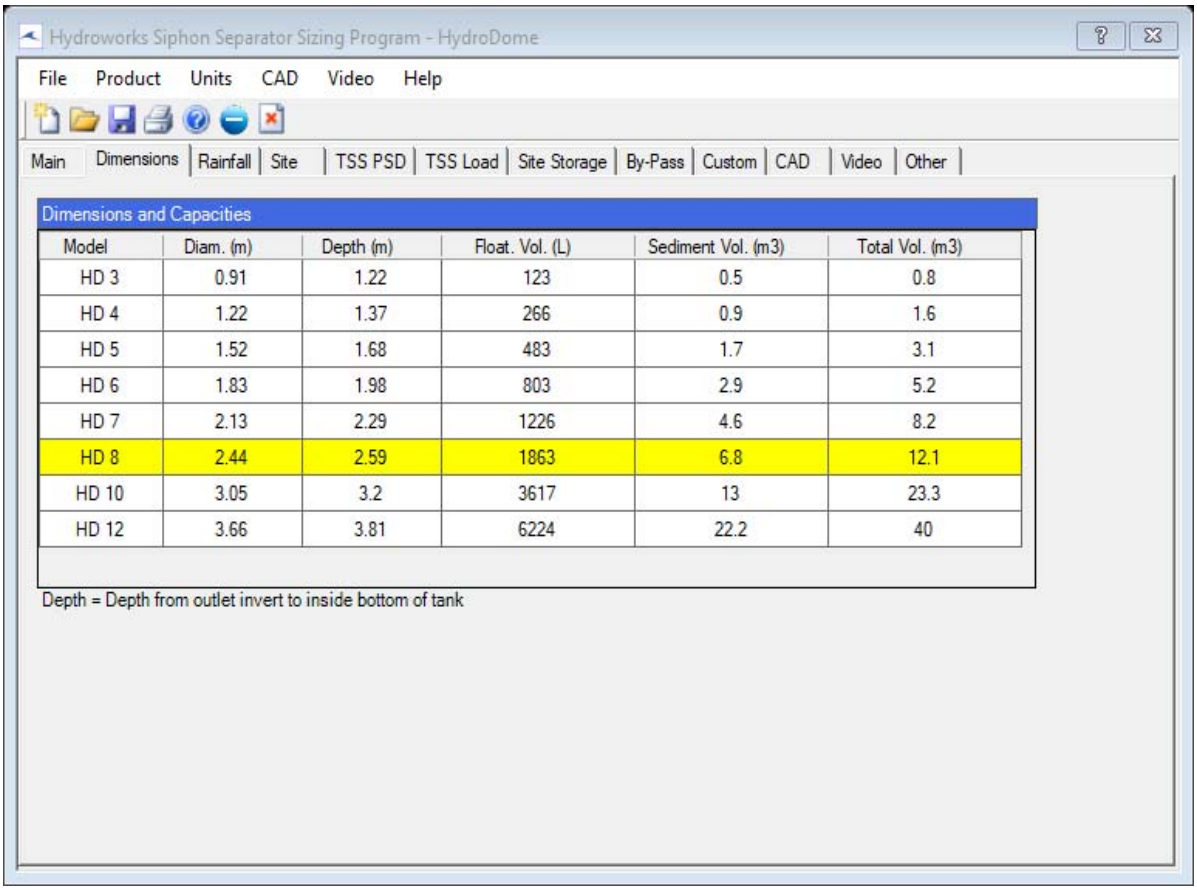
Rainfall Station - Toronto Bloor St., Ontario(1939 To 1986)



Site Physical Characteristics



Dimensions And Capacities

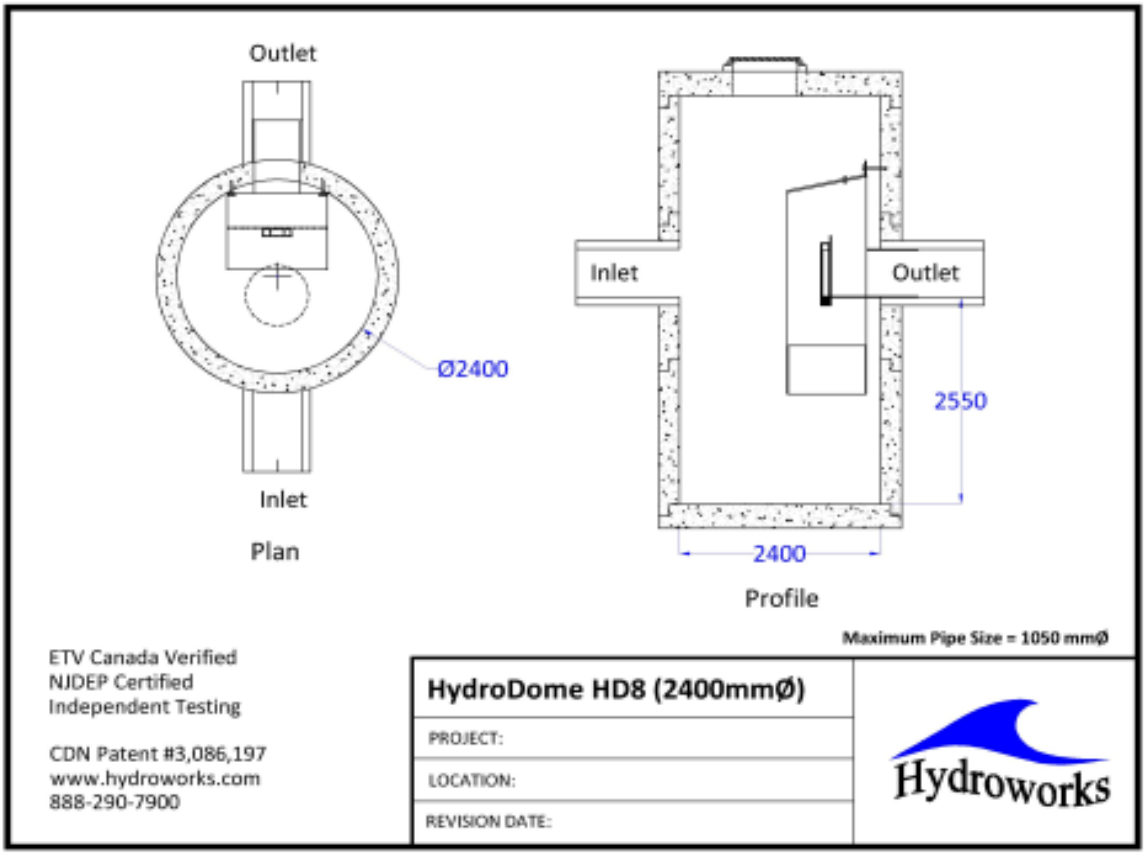


Dimensions and Capacities

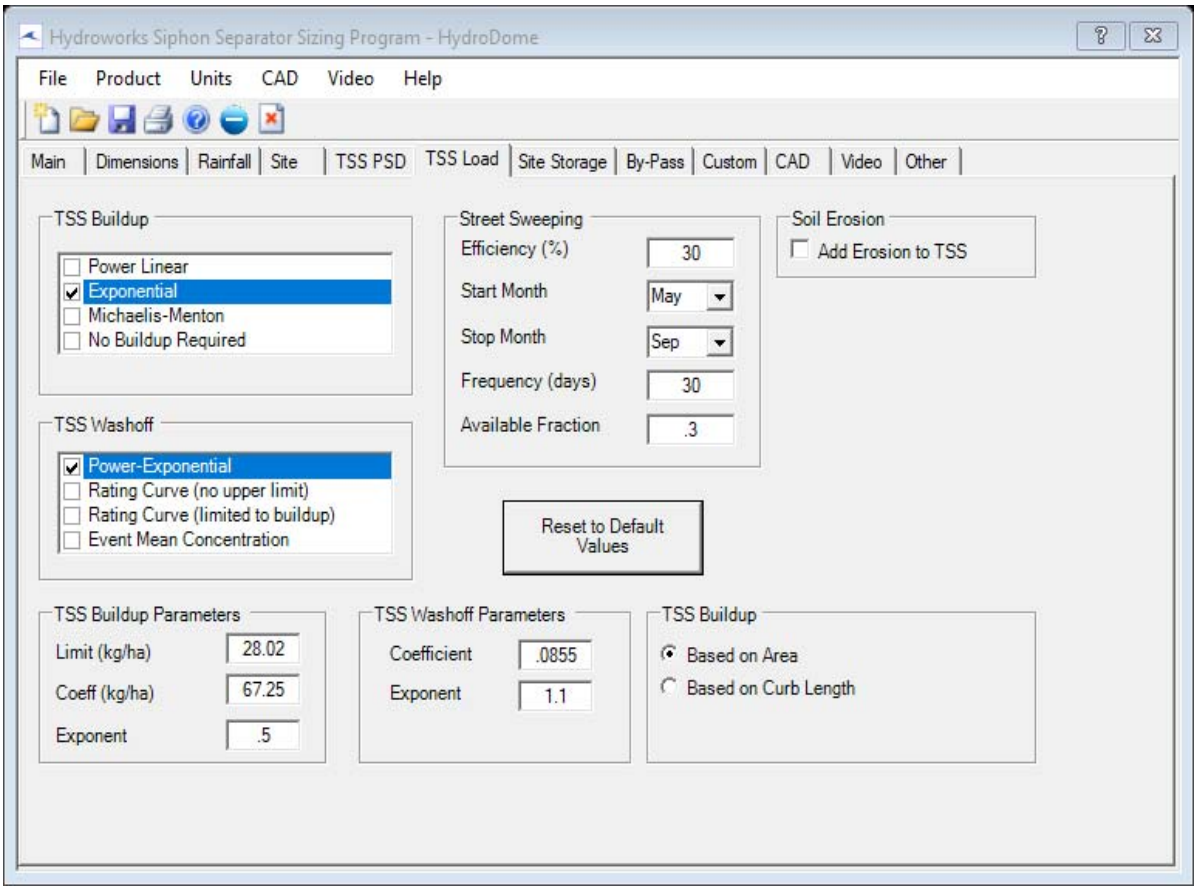
Model	Diam. (m)	Depth (m)	Float. Vol. (L)	Sediment Vol. (m3)	Total Vol. (m3)
HD 3	0.91	1.22	123	0.5	0.8
HD 4	1.22	1.37	266	0.9	1.6
HD 5	1.52	1.68	483	1.7	3.1
HD 6	1.83	1.98	803	2.9	5.2
HD 7	2.13	2.29	1226	4.6	8.2
HD 8	2.44	2.59	1863	6.8	12.1
HD 10	3.05	3.2	3617	13	23.3
HD 12	3.66	3.81	6224	22.2	40

Depth = Depth from outlet invert to inside bottom of tank

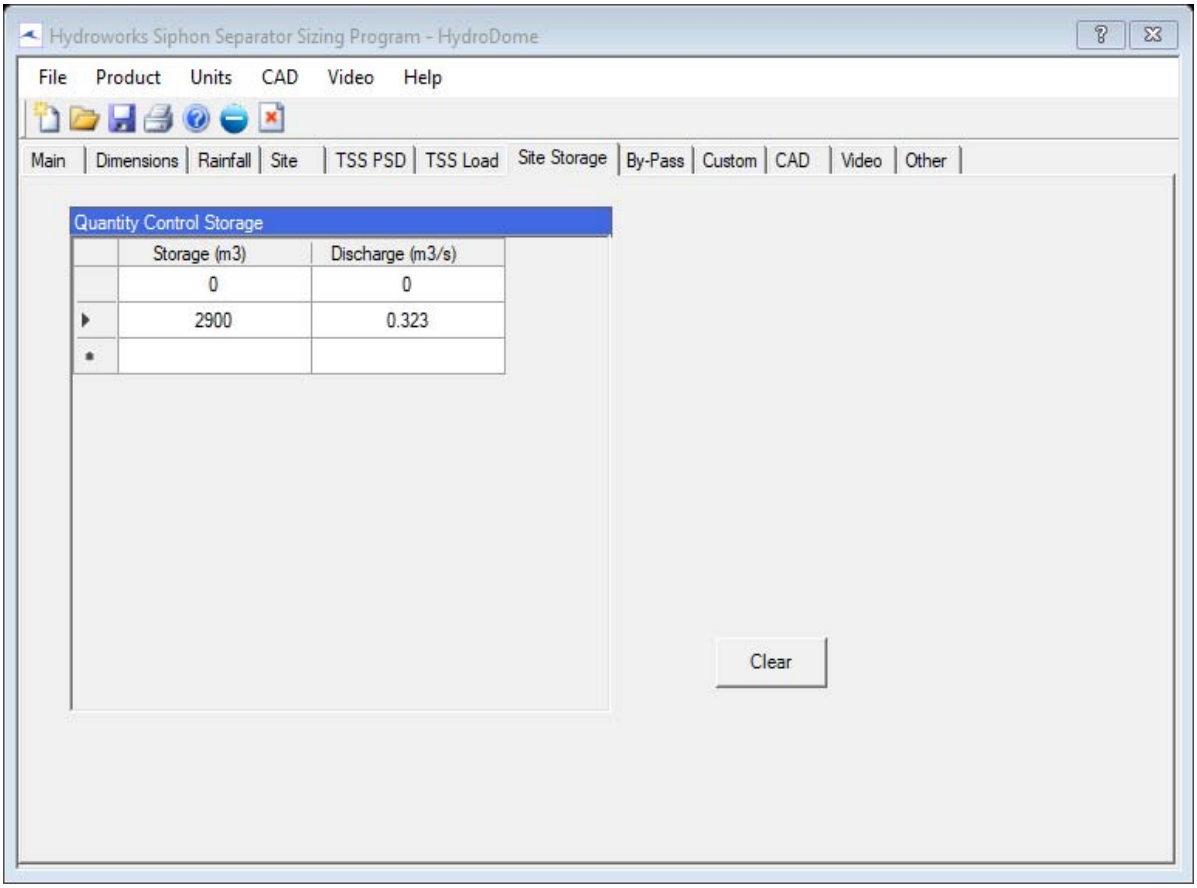
Generic HD 8 CAD Drawing



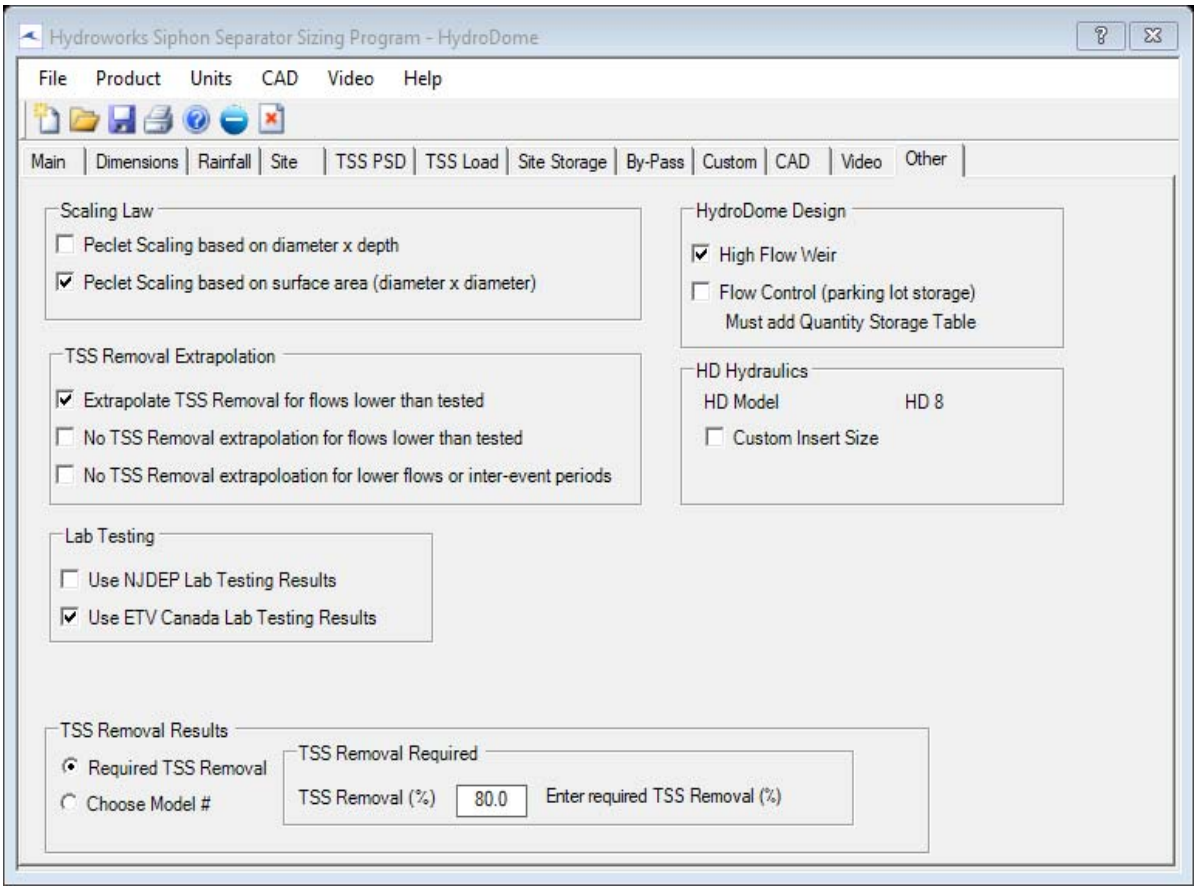
TSS Buildup And Washoff



Upstream Quantity Storage



Other Parameters

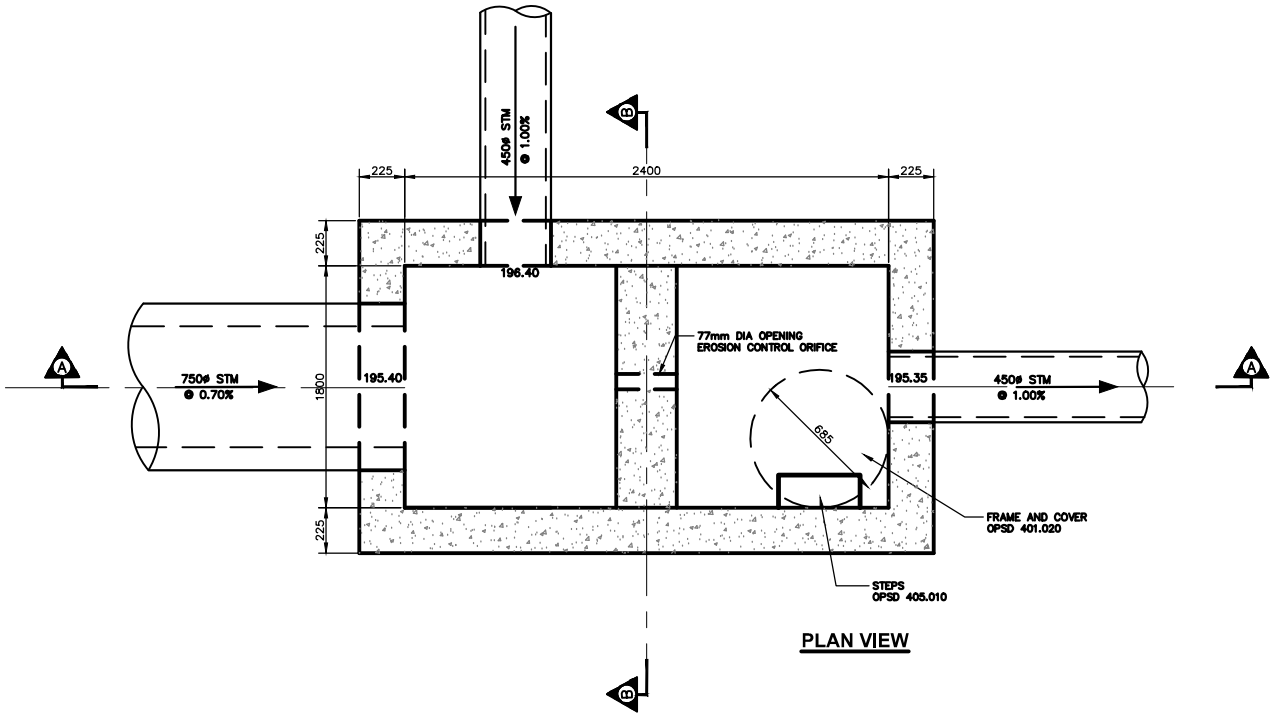


Flagged Issues

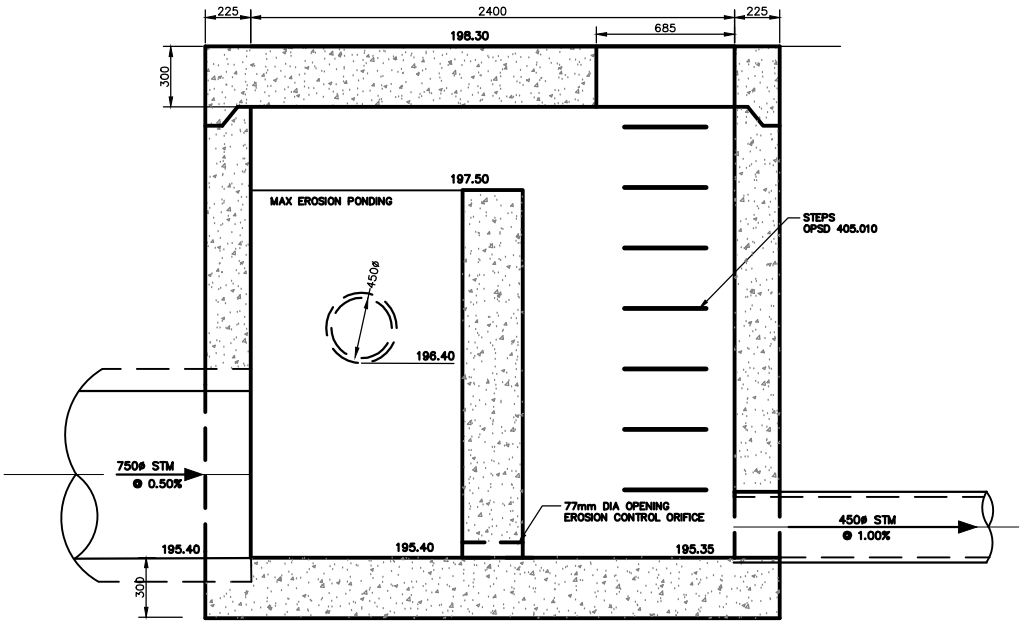
If there is underground detention storage upstream of the HydroDome please contact Hydroworks to ensure it has been modeled correctly.

Hydroworks Sizing Program - Version 5.8.5

1-800-290-7900
www.hydroworks.com



PLAN VIEW



SECTION A-A

SPLITTER MH - PRECAST 1800mmX2400mm

SCALE 1:25

JOB #: 224-M62
Prepared for: MICHAEL JOZWIK
Dezen - Vicksburg Cres
Phase 2

Proposing:
CULTEC Recharger V8 Heavy Duty H20 stormwater chambers
Units placed on 6" stone base, 6"stone above and min. 12" additional cover over for H20 application.
Units placed 60" on center. 1' stone border around perimeter of bed. Stone void calculated at 40%.

Proposed bed layout of 8 Rows x 33 Units per Row
Given:
Storage required = CF CM

STORAGE PROVIDED WITHIN CULTEC RECHARGER V8HD STORMWATER CHAMBERS

Recharger V8HD dimensions:

Width	54 inches	4.50 feet	**1.37 m**
Height	34 inches	2.83 feet	**0.86 m**
Installed Length	7.5 feet		**2.29 m**
Chamber capacity	8.933 CF/LF		**0.83 CM/LM**

Recharger V80HD Heavy Duty H20 Design Unit Capacity:

Stone base	6 inches	0.50 feet	**0.15 m**
Stone above	6 inches	0.50 feet	**0.15 m**
Center to Center Spacing	60 inches	5.00 feet	**1.52 m**
Design Unit Height	3.83 feet		**1.17 m**
Design Unit Width	5.00 feet		**1.52 m**

Number of Recharger V8HD by design =		264 pcs	**264.00 pcs**
264 pcs x 7.5' =		1980 LF	**603.50 m**
Number of Rows =		8 rows	**8.00 rows**
Total LF of chambers =		1980 LF	**603.50 m**
1980 ' x 8.933 CF/LF =		**17687.3 CF**	**500.55 CM**

STORAGE PROVIDED WITHIN CULTEC HVLV V-8 HEADER SYSTEM

CULTEC HVLV V8 Header System, Single Feed
HVLV V8 dimensions:

Width	54 inches	4.50 feet	**1.37 m**
Height	34 inches	2.83 feet	**0.86 m**
Installed Length	4.58 feet - S/E	3.33 feet - I	
Chamber capacity	8.933 CF/LF		**0.83 CM/LM**

HVLV F110x2 Feed Connector dimensions:

Width	27.5 inches	2.29 feet	**0.70 m**
Height	12 inches	1.00 feet	**0.30 m**
Installed Length	0.5 feet		**0.15 m**
Chamber capacity	1.968 CF/LF		**0.18 CM/LM**

HVLV V8 Header, Single Feed Design Unit Capacity:

Stone base	6 inches	0.50 feet	**0.15 m**
Stone above	6 inches	0.50 feet	**0.15 m**
Design Unit Height	3.83 feet		**1.17 m**
Design Unit Width	5.00 feet		**1.52 m**

Unit utilizes HVLV F110x2 Feed Connector Feed Lines on one side of Main Header

Number of Single Feed HVLV V8 Starters + Ends by design =		16 pcs	**16.00 pcs**
16 pcs x 4.58' =		73.28 LF	**22.34 m**
73.28 ' x 8.933 CF/LF =		**654.61 CF**	**60.81 CM/LM**

Calculated by:
CULTEC, Inc.
PO Box 280
Brookfield, CT 06804
PH: 203-775-4416
FX: 203-775-1462
www.cultec.com
custservice@cultec.com

Number of Single Feed HVLV V8 Intermediates by design =	0 pcs		0.00 pcs
0 pcs x 3.33' =	0 LF		0.00 m
0 ' x 8.933 CF/LF =	0.00 CF		14.17 CM/LM
Number of HVLV F110x2 Feed Connectors by design =	0 pcs		0.00 pcs
0 pcs x 0.5' =	0 LF		0.00 m
0 ' x 1.968 CF/LF =	0.00 CF		0.00 CM/LM
Storage provided within HVLV Header System alone =	654.61 CF		18.53 CM

STORAGE PROVIDED WITHIN ENTIRE CULTEC STORMWATER SYSTEM - including stone

Bed width	41.5 feet		12.65 m
Bed length	258.66 feet		78.84 m
Bed depth	3.83 feet		1.17 m
Total CF of effective excavated area	41148.5 CF		1164.50 CM
Total min. excavated area	12542.06 CF		354.94 CM
Total CF volume of HVLV Header & Recharger Chambers =		18341.95 CF	519.08 CM
Total stone required =	22806.54 CF		645.43 CM
	844.69 CY		
Storage provided within stone =	9122.618 CF		258.17 CM

Total storage within CULTEC Stormwater System = 27464.57 CF / 777.25 CM

MATERIALS LIST

MODEL	QUANTITY
Recharger V8 IHD Intermediate Heavy Duty	264
HVLV V8 SHD Starter	8
HVLV V8 IHD Intermediate	0
HVLV V8 EHD End	8
HVLV F110x2 Feed Connector	0
12.5' x 360' CULTEC No. 410 Filter Fabric	6

Calculated by:
CULTEC, Inc.
PO Box 280
Brookfield, CT 06804
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FX: 203-775-1462
www.cultec.com
custservice@cultec.com

APPENDIX D
WATER BALANCE CALCULATIONS

Project Name: DeZen Industrial Lands
Municipality: City of Mississauga
Project No.: 220-M10

Prepared by: MJ
Checked by:
Date: 13-Apr-20

Pre- and Post-Development Monthly Water Balance to Fletcher's Creek Wetland
EXISTING Condition - No Additional External Drainage Area
Located within DeZen Industrial Lands

Post-Development Scenario Desription

Area	Drainage Destination
Phase 1	Derrycrest Drive
Phase 2	Wetland
External	Wetland
Clean Rooftops	None

Winter Months not Considered

Landuse		Area (m^2)	Imperviousness	Imp. Area Runoff (m^3/year)	Perv. Area Runoff (m^3/year)	Perv. Area Recharge (m^3/year)	Total Runoff (m^3/year)	Total Recharge (m^3/year)
Pre-Development Conditions								
Hydro One Lands (Ext. 1)		25500	0.25	2460	500	167	2960	167
Open Space (Ext. 2, Ext. 4, Ext. 5, Ext. 6)		86300	0.00	0	2257	752	2257	752
Open Space/Hurontario (Ext. 3)		8800	0.43	1460	131	44	1591	44
Internal Catchment (1+2)		32900	0.00	0	860	287	860	287
Internal Catchment (3+4)		0	0.00	0	0	0	0	0
Internal Catchment (5)		0	0.00	0	0	0	0	0
		153500					**7669**	**1249**
		15	***ha***					
Post-Development Conditions								
Power Station (Ext. 1)		25500	0.25	2460	500	167	2960	167
Open Space (Ext. 2, Ext. 4, Ext. 5, Ext. 6)		86300	0.00	0	2257	752	2257	752
Open Space/Hurontario (Ext. 3)		8800	0.43	1460	131	44	1591	44
Internal Industrial (Phase 1)	uncontrolled	1652	0.25	159	32	11	192	11
Internal Industrial (Phase 2)	uncontrolled	4852	0.25	468	95	32	563	32
Roof Drainage (none)			1.00	0	0	0	0	0
Total Post-Development		**127104**					**7563**	**1005**
		12.7	***ha***					

TOTAL PRE-DEVELOPMENT VOLUME TO WETLAND **8,918** m^3/year

TOTAL POST-DEVELOPMENT VOLUME TO WETLAND **8,568** m^3/year

TOTAL VOL CHANGE FROM PRE- TO POST-: **350** m^3/year

Percent Increase Volume = (Total Post Vol. / Total Pre Vol.) x 100 **96 %**

RUNOFF VOL CHANGE FROM PRE- TO POST-: **105** m^3/year ***Decrease***

RECHARGE VOL CHANGE FROM PRE- TO POST-: **244** m^3/year ***Decrease***

Project Name: DeZen Industrial Lands
Municipality: City of Mississauga
Project No.: 220-M10

Prepared by: MJ
Checked by: 0
Date: 13/04/2020

Pre- and Post-Development Monthly Water Balance to Fletcher's Creek Wetland
EXISTING Condition - No Additional External Drainage Area

Located within DeZen Industrial Lands

Winter Months Considered

Landuse	Area (m^2)	Impervious-ness	Imp. Area Runoff (m^3/year)	Perv. Area Runoff (m^3/year)	Perv. Area Recharge (m^3/year)	Total Runoff (m^3/year)	Total Recharge (m^3/year)
Pre-Development Conditions							
Hydro One Lands (Ext. 1)	25500	0.25	4245	3469	1156	7714	1156
Open Space (Ext. 2, Ext. 4, Ext. 5, Ext. 6)	86300	0.00	0	15652	5217	15652	5217
Open Space/Hurontario (Ext. 3)	8800	0.43	2520	910	303	3429	303
Internal Catchment (1+2)	32900	0.00	0	5967	1989	5967	1989
Internal Catchment (3+4)	0	0.00	0	0	0	0	0
Internal Catchment (5)	0	0.00	0	0	0	0	0
	153,500					**32,762**	**8,666**
	15.350	***ha***					
Post-Development Conditions							
Power Station (Ext. 1)	25500	0.25	4,245	3,469	1,542	7,714	1,542
Open Space (Ext. 2, Ext. 4, Ext. 5, Ext. 6)	86300	0.00	-	15,652	5,217	15,652	5,217
Open Space/Hurontario (Ext. 3)	8800	0.43	2,520	910	532	3,429	532
Internal Industrial (Phase 1)	1652	0.25					
Internal Industrial (Phase 2)	4852	0.25	808	660	293	1,468	293
Roof Drainage (none)	0	1.00	-	-	-	-	-
Total Post-Development	**127,104**					**28,263**	**7,584**
	12.7	***ha***					

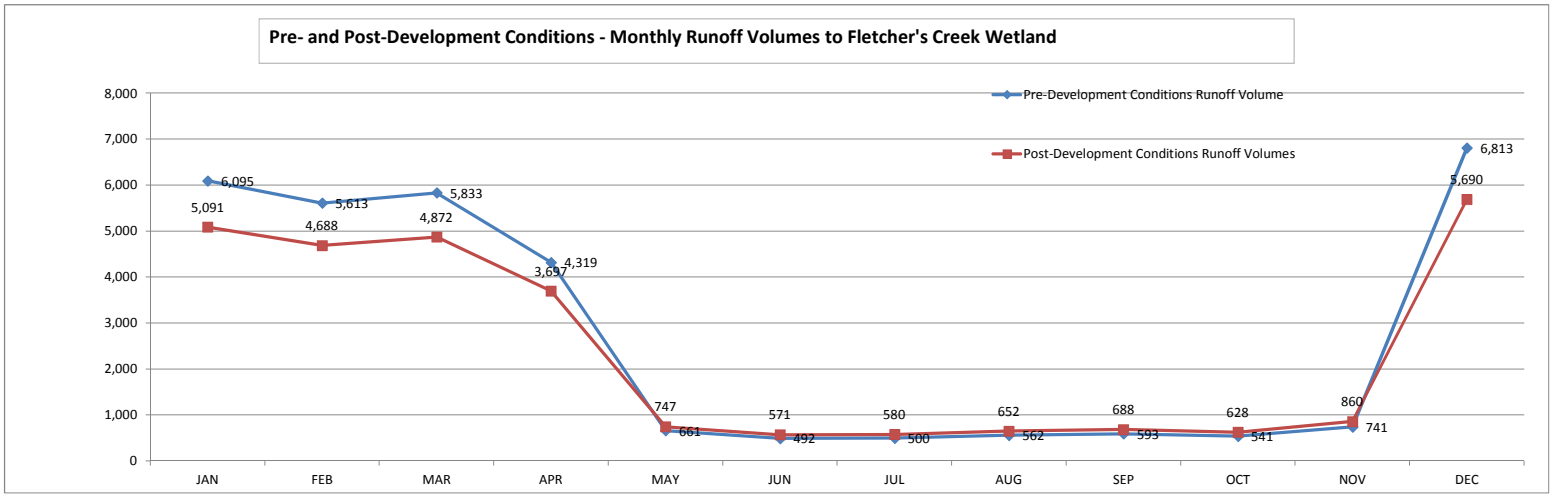
TOTAL PRE-DEVELOPMENT VOLUME TO WETLAND	**41,428**	**m^3/year**		
TOTAL POST-DEVELOPMENT VOLUME TO WETLAND	**35,847**	**m^3/year**		
TOTAL VOL CHANGE FROM PRE- TO POST-:	**5,581**	**m^3/year**	**Percent Increase Volume = (Total Post Vol. / Total Pre Vol.) x 100**	**87 %**
RUNOFF VOL CHANGE FROM PRE- TO POST-:	**4,499**	**m^3/year**	***Decrease***	
RECHARGE VOL CHANGE FROM PRE- TO POST-:	**1,081**	**m^3/year**	***Decrease***	

Pre-Development Conditions Monthly Runoff

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Monthly Depth (m) - Shortrooted	0.039	0.036	0.037	0.026	0.000	0	0	0	0	0	0.000	0.043	**0.181**
Area (m^2) - Shortrooted	143,341												153,500
Runoff Volume (m^3) - Shortrooted	5,569	5,128	5,328	3,692	56	0	0	0	0	0	0	6,225	**25,997**
Impervious Monthly Depth (m)	0.052	0.048	0.050	0.062	0.060	0.048	0.049	0.055	0.058	0.053	0.073	0.058	**0.666**
Impervious Area (m^2)	10,159												
Impervious Area Volume (m^3)	526	485	505	626	605	492	500	562	593	541	741	588	**6,765**
Pre-Development Conditions Total Runoff (m^3)	***6,095***	***5,613***	***5,833***	***4,319***	***661***	***492***	***500***	***562***	***593***	***541***	***741***	***6,813***	***32,762***

Post-Development Conditions Monthly Runoff

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Monthly Depth (m) - Shortrooted	0.039	0.036	0.037	0.026	0.000	0	0	0	0	0	0.000	0.043	**0.181**
Area (m^2) - Shortrooted	115,319												
Runoff Volume (m^3) - Shortrooted	4,480	4,126	4,286	2,971	45	0	0	0	0	0	0	5,008	**20,915**
Impervious Monthly Depth (m)	0.052	0.048	0.050	0.062	0.060	0.048	0.049	0.055	0.058	0.053	0.073	0.058	**0.666**
Impervious Area (m^2)	11,785												
Impervious Area Volume (m^3)	610	562	586	727	702	571	580	652	688	628	860	682	**7,847**
Post-Development Conditions	***5,091***	***4,688***	***4,872***	***3,697***	***747***	***571***	***580***	***652***	***688***	***628***	***860***	***5,690***	***28,762***



	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
Pre-Development Conditions Total Runoff (m3)	*6,095*	*5,613*	*5,833*	*4,319*	*661*	*492*	*500*	*562*	*593*	*541*	*741*	*6,813*
Post-Development Conditions Total Runoff (m3)	*5,091*	*4,688*	*4,872*	*3,697*	*747*	*571*	*580*	*652*	*688*	*628*	*860*	*5,690*
Volume Change m3 per month	**-1,004**	**-925**	**-961**	**-622**	**86**	**79**	**80**	**90**	**95**	**87**	**119**	**-1,123**
Total Runoff Volume (Post / Pre x 100)	84%	84%	84%	86%	113%	116%	116%	116%	116%	116%	116%	84%

Note: Water balance calculations were determined using the Thornwaite and Mather methodology (industry standard). The Thornwaite and Mather methodology was developed as a simple "bookkeeping" model that uses total monthly precipitation values and estimates for potential evapotranspiration and taking into account an initial estimate for soil storage to approximate the monthly water balance. Soil moisture storage in these calculations are based on vegetation type (i.e. short-rooted - grass approx. 100 mm depth). This methodology is used to evaluate water volume inputs to streamflow and/or wetlands. The water balance analysis estimates the starting soil moisture storage for each month (carried over from the last month), adds the precipitation and subtracts the losses from evapotranspiration. It is important to remember that this is a balance calculated on a monthly period with net total volume results as the output of the water balance.

APPENDIX E

SANITARY DESIGN SHEET
SANITARY TRUNK DRAWINGS

SUBDIVISION : **DEZEN CONSTRUCTION COMPANY LIMITED**

REGION FILE:

CONSULTANT : **SKIRA & ASSOCIATES LTD.**

REGION OF PEEL

SANITARY SEWER DESIGN CHART

SHEET No. 1 of 1

PROJECT No. : 220-M10

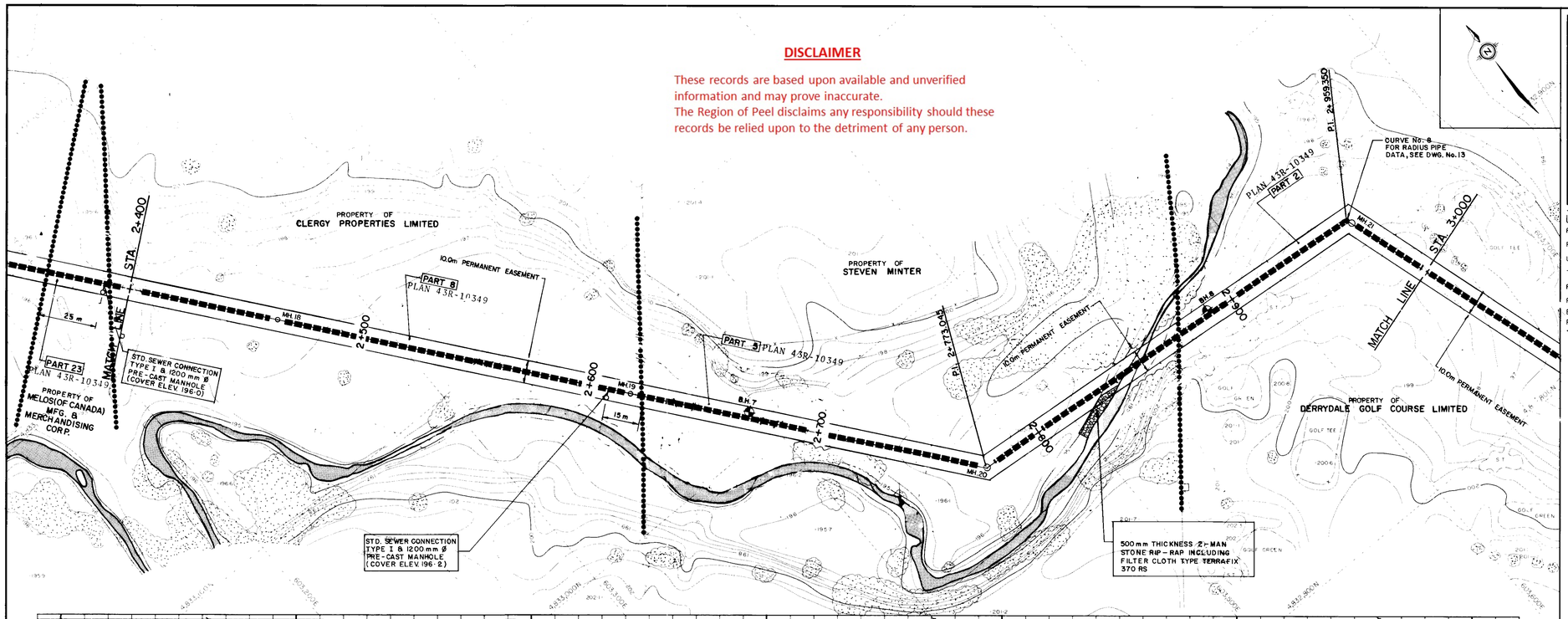
DESIGNED BY : M.J.

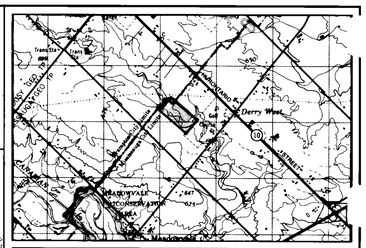
DATE : APR.2020

LOCATION	FROM MH	TO MH	AREA Aa	DENSITY	POP. Pp	ACCUM. AREA A=∑Aa	ACCUM. POP. P=∑Pp	SEWAGE FLOW	INFILT. FLOW	TOTAL FLOW Q_{ACT}	TYPE OF PIPE	LENGTH L	SLOPE S	PIPE SIZE NOMINAL D	CAPACITY n=0.013 Q_D	VELOCITY n=0.013 V	VELOCITY ACTUAL Va	INVERT ELEV. UPPER	INVERT ELEV. LOWER
	MH#	MH#	ha	ppha		ha		m^3/s	m^3/s	m^3/s		m	%	mm	m^3/s	m^3/s	m^3/s	MH	MH
VICKSBURGH DR	EX.3A	EX.2A	4.99	70	349	4.99	349	0.013	0.0010	**0.014**	PVC	20.0	0.35	300	**0.060**	0.82			
PHASE 1	EX. MH	EX.2A	3.85	70	270	8.84	619	0.013	0.0018	**0.015**	PVC	35.7	0.28	300	**0.053**	0.73			
	EX.2A	EX.1A	0.25	0	0	9.09	619	0.013	0.0018	**0.015**	PVC	88.0	0.36	300	**0.061**	0.83			
	EX.1A	EX.SAN	1.14	70	80	10.23	699	0.013	0.0020	**0.015**	PVC	77.5	0.39	300	**0.063**	0.86			
PHASE 2	EX.20	SAM.MH	7.51	70	526	7.51	526	0.013	0.0015	**0.015**	PVC	95.0	1.00	250	**0.062**	1.22			

DISCLAIMER

These records are based upon available and unverified information and may prove inaccurate.
The Region of Peel disclaims any responsibility should these records be relied upon to the detriment of any person.





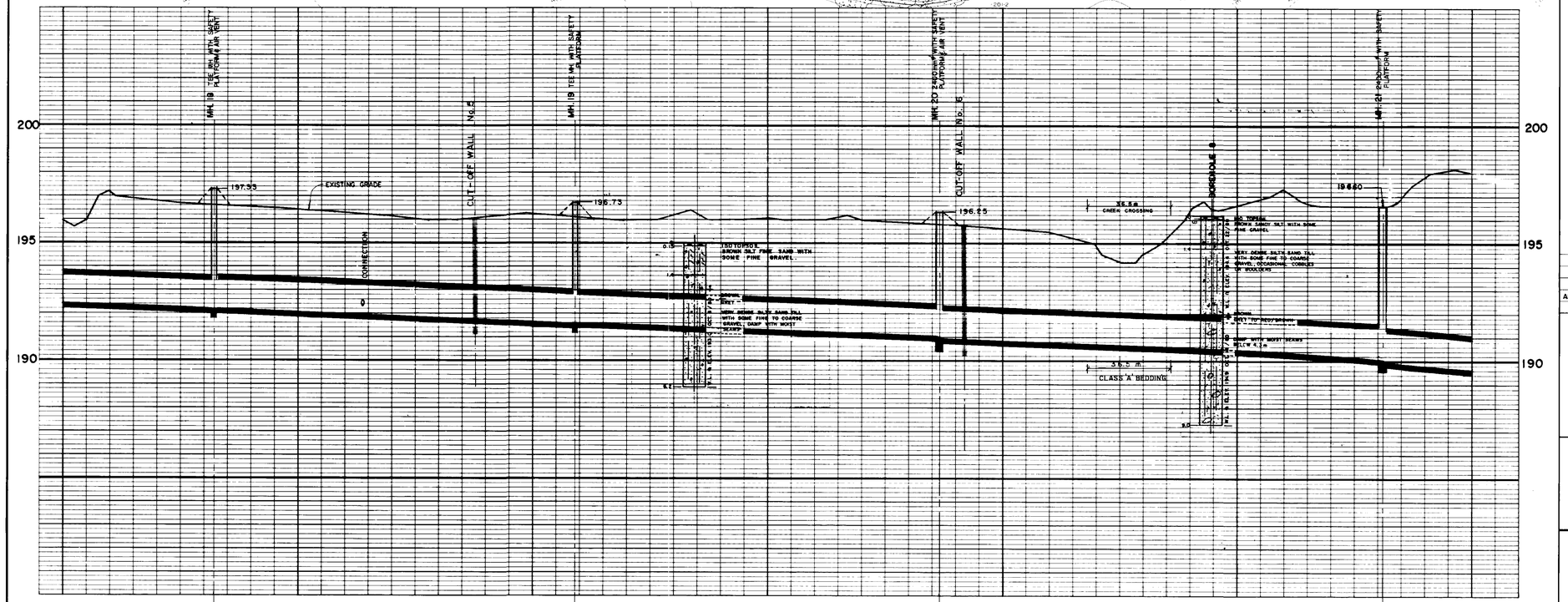
LEGEND

RAILROAD		PROPERTY LINE	
PAVED ROAD		BUILDING	
UNPAVED ROAD		WOODED AREA	
TRACK OR TRAIL		RIVER	
FENCE	x	STREAM, INDEFINITE	
POLE		CONTOURS	390
B" REHOLE	B.H.	SPOT ELEVATION	396.2
PERMANENT EASEM'T.			

Topographical maps produced by KENTING EARTH SCIENCES LIMITED, TORONTO, from Aerial Photography Dated MAY 1980.

Geodetic Datum (1927 North American)
Contour Interval 1 Metre

Transverse Mercator Projection 6° Zone 17



SEWER DATA	153.9 m-1200mm ϕ R.C. PIPE, CLASS III @ 0.37 %		153.9 m-1200mm ϕ R.C. PIPE, CLASS III @ 0.32 %		156.5 m-1200mm ϕ R.C. PIPE, CLASS III @ 0.40 %		189.2 m-1200mm ϕ R.C. PIPE, CLASS IV @ 0.41 %		118.5m-1200mm ϕ R.C. PIPE, CLASS V @ 1.05 %
INVERT ELEV.	192.117		191.626		190.996	190.900		190.126	189.939
STATION	2+400	2+500	2+600	2+700	2+800	2+900	3+000		

FOR GENERAL NOTES & BENCH MARK INFORMATION, REFER TO DRAWING NO. 1.

Date	Revision	Init
APRIL, 1984	AS CONSTRUCTED	N.D.

DESIGNED BY

APPROVED BY

REGISTERED PROFESSIONAL ENGINEER N. C. DEVANI PROVINCE OF ONTARIO

Region of Peel

Department of Public Works

PROJECT NO. 82-2113

kleinfeldt consultants limited
CONSULTING ENGINEERS AND PLANNERS
4 MELANIE DRIVE, SUITE 12, BRAMPTON, ONTARIO L6T 4L1

FLETCHER'S CREEK SANITARY TRUNK SEWER
(NORTH SECTION)
PLAN & PROFILE
STA 2+400 TO STA 3+000

Designed by N.C.D.	Date		Project No. 1-1802
Drawn by D.B.S.	SCALE		DWG. No.
Chk'd by N.C.D.	Horizontal	1:1,000	5
Approved by I. D.	Vertical	1:100	

D-3316

91/90-D

APPENDIX F

STORMWATER MANAGEMENT REPORT
BY SERNAS

SERNAS ASSOCIATES
A Member of The Sernas Group Inc.

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Mississauga, ON
L4Z 1X3

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F·905·890·8499
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April 4, 2003

City of Mississauga
Transportation and Works Dept.
Environmental Division
3484 Semenyk Court
Mississauga, Ontario
L5C 4R1

Attention: Mr. Brian Chan,
Coordinator Infrastructure and Environmental Planning

Re: Stormwater Management Facility
Flow Control Performance Monitoring
Operation and Maintenance Manual
SWM Facility 4402B - Fletcher's Creek Business Park
City of Mississauga
Our Project No. 02105.400

Enclosed are five (5) copies of the Operation and Maintenance manual for the Stormwater Management Facility No. 4402B at Derry Rd. and Hurontario St.

Please do not hesitate to contact us with any questions regarding the enclosed or above information.

Yours truly,

SERNAS ASSOCIATES

for Ken Chow, P.Eng.
Principal, Manager – Water Resources

KC: ma

Encl.

c.c. Sernas Associates, Attn: Muneef Ahmad

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FIGURES

APPENDICES

1.0 INTRODUCTION

The City of Mississauga has retained Semas Associates to perform Maintenance Monitoring as part of a Flow Control Performance Monitoring study on four Stormwater Management facilities within their jurisdiction. This Operation and Maintenance Manual is for Stormwater Management (SWM) Facility No. 4402B: Fletcher's Creek Business Park.

The objective of the monitoring study was to ensure that the SWM facilities operate effectively and efficiently. In addition to the Performance Monitoring, it was desired to investigate the condition of the SWM facilities in order to:

- Establish a monitoring program
- Establish a maintenance schedule with associated costs
- Estimate the accumulated sediment

From the investigation, recommendations will be made if required to the operating and maintenance characteristics of the facilities.

Section 2 of the report will outline the background for the study as well as the procedures implemented in the monitoring program.
Section 3 will outline the Operation of the SWM facility.
Section 4 will document the recommended Maintenance procedures for the facility with estimated costs.
Section 5 will summarize Conclusions and Recommendations.

2.3 SEDIMENT ACCUMULATION

Sediment accumulation within the forebay was determined by rod measurements in the forebay. A canoe was launched in the forebay for sediment depth measurement with the rod used to determine the top of sediment depth at various pre-mapped locations. Relative elevations were correlated by using the geodetic water level measurements established as part of the Flow Control Performance Monitoring portion of the overall study. Finally, sediment volumes were calculated based on the geometry of the SWM facility forebay and depth of sediment versus the design bottom elevation.

Bathymetric measurements refer to the measurement of depths of bodies of water and in this case refer to the survey and mapping of the stormwater management facility below water. Further discussion is also included in Section 4.4 on Alternative Survey Methods for SWM Facility Bathymetry.

2.4 MAINTENANCE MONITORING & SITE INVENTORY

A detailed visual review was undertaken at the commencement of the assignment to determine the existing condition of the inlet pipes, bypass structures, forebay, berms, outlet control structure, erosion within, around and downstream of the SWM facility, health of existing vegetation within and adjacent to the SWM facility, existence of oil or other spilled material, and a general review of the site from a safety and security viewpoint. A monitoring checklist was created to assist in the assessment and the results will be summarized and included in the final report.

2.5 EROSION

The lands downstream of the facility have been monitored for both erosion and sedimentation of eroded material. Facility 4402B outlets to the Fletcher's Creek. Stakes were placed in the outlet channel at 15m intervals to the property limit. These stakes have been monitored on a bi-monthly basis to determine if downstream erosion is occurring or, if sedimentation is taking place. The outlet channel can be described as shallow swale with moderate growth in the warm season.

3.0 OPERATION

FACILITY No. 4402B: Fletcher's Creek Business Park

The stormwater management (SWM) facility is shown on Cosburn Patterson Mather Limited Drawing SW1 enclosed in the rear of the report.

The facility has been designed as a quality/quantity control facility that will provide Level 1 quality protection and attenuate post development runoff flows to predevelopment release rates for the 2 year to 100 year storm events from a catchment area of approximately 129 hectares. The facility has been designed to provide 17,120 m^3 of permanent pool volume between an elevation of 187.5 m and 190.5 m. The extended detention component will provide a storage volume of 11,180 m^3 up to an elevation of 191.55 m. The design top of pond is set by a concrete weir at an elevation of 193.50 m, where a total of 40,130 m^3 of active storage is provided.

The quality portion of the SWM facility has been designed to remove some sediments and pollutants from the stormwater runoff from the contributing area. For particles to settle to the bottom of the pond a dewatering time greater than 24 hours is required. To achieve a dewatering time of 24 hours or greater an orifice type control structure was utilized. The outlet has been designed utilizing a 750 mm diameter reverse sloped pipe with a 550 mm orifice located in a control manhole.

Release rates resulting form flows associated with events greater than the 25mm storm up to the 100 year event, are controlled by a concrete weir located at the south west corner of the facility. The two meter wide weir has a top elevation of 192.0 m, which is below the water level corresponding to the 2 year event.

All flows including the release from the outlet structure and flows in excess of the pond capacity, which over top the weir, are diverted to Fletcher's Creek.

A gravity drain below the permanent pool elevation has not been provided. As such, the pond will have to be pumped down below the permanent pool elevation of 190.5 m for facility maintenance and cleaning purposes.

4.0 MAINTENANCE

The manual will include a checklist sheet to be used by an inspector, inspecting the facility. It will outline maintenance requirements for the SWM facility such as fences, gates, vegetation, erosion concerns, debris, clogging areas and removal of sedimentation. Clean out calculations and frequency of clean out will also be provided.

4.1 VISUAL INSPECTION GUIDELINES

Site inspection personnel should visually monitor the operations of the quality pond and the control structure every 6 months. The facility shall be inspected on a routine basis for the items listed below, as a minimum to identify required corrective actions:

- Clogging of flow splitting, inlet and outlet structures.
- Presence of litter, debris or other foreign material.
- Water clarity, oily material or evidence of any upstream spills.
- Safety and security measures are in place and functioning (i.e., fences, gates, etc).
- Erosion of internal and external berms or around structures. Evidence of erosion at intake, outlet, berms, slopes downstream channel, overflow structures;
- Re-vegetation issues, both aquatic and terrestrial. Condition of the vegetation, whether they are healthy or require replacement or are growing normally;
- Visually obvious sediment buildup (i.e., if its obvious that the pond is filling up such that it exceeds the design requirements, then clean the pond).
- Maintenance access to the facility (i.e., ensure it is clear).
- Pond drying out or experiencing an unexpectedly fast or slow draw down time.
- Safety and security check such as fences, warnings signs, grates, steep slopes, and other hazards;

During the routine inspections, further safety related items that may be unsafe, cause damage or injury to the public should be noted, including but not restricted to:

- Slope erosion.
- Settlement along areas accessible to pedestrians.
- Vandalism.

Additional items not listed above shall be noted in the Inspector's Checklist that is attached in Appendix B.

4.2 QUALITY POND

4.2.1 MAINTENANCE GUIDELINES

Outlined are a set of procedures/guidelines, which can be followed for cleaning and restoration of the quality ponds:

Prior to commencement of cleaning the facility, inventory of the existing vegetation shall be noted

Pumping of the quality pond with the pond bottom allowed to "dry out" if the weather and time of year permit. To assist in draining water from the sediments accumulated in the quality pond, it may be worthwhile to excavate a depression to below the bottom elevation of the SWM facility after draining the facility by pumping. This will allow for water to drain out of the sediment and into the depression and dry in place without additional movement of the material.

Removal of the excess material by dragline or, if possible, with the use of a backhoe.

Placement of the material on the sides of the pond to allow drying of the material. Due to the slope, measures may have to be taken to prevent material from flowing to the bottom. Alternatively, material can be moved to a flat area via "leak proof" dump trucks.

Reshaping of the quality pond to original design.

All disturbed areas including the sediment forebay shall be restored with 100 millimeters of topsoil.

Existing trees, shrubs and aquatic herbs removed or damaged during the cleaning operation must be replaced to the satisfaction of the municipality.

4.2.2 ANNUAL SEDIMENT LOADING RATE

The annual sediment loading volume was computed using the MOEE 1994, Stormwater Management Practices Planning and Design Manual to be approximately 307 m^3. This volume was based on a catchment area of 129.0 ha, with an estimated 63% imperviousness and an annual loading rate of 2.38 m^3/ha. The formula and corresponding calculations are shown in Appendix A.

4.2.3 OBSERVED IN-SITU SEDIMENT ACCUMULATION

The sediment forebay has been designed with a capacity of approximately 2000m^3. Based on field measurements, there is a depth of less than 0.10m (on average) at the deepest area of the sediment forebay. This equates to an estimated accumulated sediment volume within the forebay of 70m^3 that equates to an annual loading volume of 0.3m^3/ha assuming the facility has been in operation for 2 years. This lower than expected loading rate results from the fact that the tributary area has not been built out to its design condition as yet.

4.2.3 SEDIMENT REMOVAL FREQUENCY

The minimum sediment removal frequency of 10 years is stated in the Maintenance section of the 1994 MOEE SWMP Manual although a value of 31 years was computed using the 1994 MOEE SWMP Guidelines. Using Table 4.1 (from the 1994 MOEE SWMP manual), with an estimated 63% imperviousness resulted in a storage volume of 210.00 m^3/ha for the SWM facility. The storage volume of 210.00 m^3/ha from Table 4.1 was used on Fig. 5.1 and 5.2 to determine the average sediment removal frequency estimated to be every 31 years with these guidelines. The corresponding calculations are shown in Appendix A.

The MOEE manual also recommends cleanout of the SWM facility when there is a 5% reduction in treatment efficiency. If loading rates were as expected, sediment removal would be required in less than 5 years as a 5% reduction in volume would occur in approximately 3 years.

Since accumulated sediment volumes are considerably low it is recommended that accumulated sediment be re-examined in 5 years or when the tributary area has been built out closer to its design condition, whichever comes first. At this point it can be determined when maintenance would be required. If the tributary area were to be built to its design condition in the near future, it is estimated that maintenance would be required in eight years (Year 2011).

4.2.4 SEDIMENT REMOVAL

The main purpose of the quality pond is to settle suspended material. The suspended material settles on the bottom of the pond and therefore must be cleaned on a routine basis. It is assumed the facility must be cleaned on an estimated cycle of every 10 years as stated above.

It is our suggestion that prior to excavation and disposal that the material be properly tested to comply with the MOEE standards. The MOEE administers the Ontario Water Resources Act of which Section 53 is applicable for construction of stormwater management facilities. The MOE issues Certificates of Approval (C of A) for construction of SWMF's and conditional in the C of A are typical clauses such as the "owner shall ensure that sediment is removed from the stormwater management works at such a frequency as to prevent the excessive buildup and potential overflow of sediment into the receiving watercourse. Additionally, the MOE administers the Environmental Protection Act that pertains to sediment disposal because it regulates the disposal of pollutants into the natural environment. As described in the *Sediment Maintenance Guide* (p.9),

"Under the Act, Regulation 347-Schedule 4, describes leachate quality criteria and analysis procedures used to determine if the contamination level of the material tested is too high and requires landfilling at a registered non-hazardous or hazardous waste management facility. In addition, registered sediment waste must pass the slump test in order to qualify as solid waste rather than as liquid waste. Sediment with high liquid content must be dewatered if it is to be accepted at a solid waste disposal facility."

Other criteria from the MOEE that may be applicable are:

- Guidelines for Clean Up of Contaminated Sites in Ontario, bulk sediment analysis criteria
- Guidelines for the Protection and Management of Aquatic Sediment Quality in Ontario
- Provincial Water Quality Objectives

If the material is considered solid non-hazardous waste, then disposal at a landfill site will be acceptable. Landfill sites have their own disposal rates and the cost varies depending on such factors as: the region in which the site is located, type of waste, quantity, etc.

Detailed discussions of sediment chemistry have not been addressed as the expected chemistry from residential catchments have lower contaminant concentrations. Typically this will not require landfill disposal. The *Sediment Maintenance Guide* is referred to for further information on the needs and methodology for checking sediment chemistry.

The testing would consist of one or two samples being tested by an approved lab. The samples would be tested for excess quantities of substances not acceptable for landfill site disposal. The quantity and type of non-acceptable material varies between landfill sites. Presently, the cost for sampling and testing for hydrocarbons, PCB's, ignitable and inorganic material is approximately $600 per sample.

As the vegetation in the detention facility matures, a successional type growth will occur. The clean out of the bottom of the sediment forebay would not adversely affect most of the bank vegetation, but will disturb existing growth at the bottom of the pond. It is our belief that by leaving small sections or clumps of the existing vegetation, it will grow over the entire bottom of the facility quite quickly.

4.3 EROSION

No erosion or sedimentation was found in the downstream channel. This is based on the fact that the facility has not reached its design permanent water level as yet and thus rarely experiences outflow. As outlined in the original proposal, the stakes will be left in place at the City's request for future erosion monitoring.

4.4 ESTIMATED MAINTENANCE COSTS

Attached in Appendix 'C' is a cost estimate for maintenance and sediment removal of Facility No. 4402B that details the $98,000 value (Cdn. 2003). The following assumptions were made in preparing the estimate utilizing information gathered from the Flow Control Performance Monitoring Study, MOEE SWMP Manual and the *Sediment Maintenance Guide*:

- Facility commenced operation approximately 2 years ago and has never been cleaned
- Facility will be pumped to dewater
- No special sediment dewatering procedures are required
- Sediment removal will be done with dozer and mechanical excavator to load dump trucks
- Diversion of runoff during rain events will be done with continuous pumping to bypass the facility
- All affected vegetation will be completely removed and re-instated, however no significant aquatic vegetation is provided/required

The final cost can vary substantially from the estimate due to site-specific factors that result from the maintenance operation. Factors that substantially affect cost are listed below:

- Method of drying sediment (i.e. bulking, air dried, etc.)
- Relative distance to disposal site
- Amount of restoration required after completion
- Costs related to dealing with sediment contamination (not highly anticipated with residential runoff)
- Landfill charges
- Alternative dredging methods such as hydraulic techniques are available but may not be cost-effective for every scenario. The choice of technology depends on several factors such as: site accessibility, volume to be removed, options to by-pass, inflow runoff, in-situ sediment drying potential.

4.5 ALTERNATIVE SURVEY METHODS FOR SWM FACILITY BATHYMETRY

As documented in the Sediment Maintenance Guide, there are several standard methods for surveying below water:

- Core sampling: a special tube is used to collect samples from the bottom of the facility whereby the consistency of the material in the sample is used to estimate the sediment depth and thus identify the depth at each sample location

- Disc/rod measurements: employs a disk attached to a rod which rests on top of the sediment, followed by a measurement with only the rod that penetrates the looser sediment to a firmer base that is assumed to be the bottom of the facility. Comparing the two measurements results in sediment depth estimation

- Depth sounding: "Fishfinder" devices that utilizes sonar (SOund Navigation And Ranging) technology to illustrate the bottom profile of the SWM facility. With the information pertaining to the bottom of the SWM facility, it can be compared to design information to allow for estimation of an accumulated sediment depth and an associated sediment volume.

The technologies described above are acceptable and used in general practice. However, these methods can be labour intensive and in the instances of core sampling and disc/rod measurements can result in under or over estimation of accumulated sediment due to inherent disadvantages with each strategy.

Based on sediment loading rates described in the MOE manual, annual sediment accumulation in a SWM facility could alter the bottom elevation of a facility anywhere from 0.25m to less than 0.05m. The variance is based on the size of the settling area in the SWMF and the expected amount of sediment from varying levels of imperviousness in the tributary area.

Accordingly, it was felt that a more accurate measure of sediment accumulation rates on a watershed basis could be obtained by surveying SWM facility bathymetry over a course of several years.

An alternative method of measurement was explored as a potential option and the results are documented in this report.

LIDAR is an acronym for LIght Detection And Ranging. Where a remote target exists, either a clearly defined object, such as a vehicle, or a diffuse object such as a smoke plume or clouds, LIDAR has the ability to measure:

- Distance
- Speed
- Rotation
- Chemical composition and concentration

Generally there are three types of LIDAR:
- Range finders are used to determine distance from the source to target
- DIAL (DIfferential Absorption LIDAR) is used to measure chemical concentration in the atmosphere
- Doppler Lidars are used to measure the velocity of a target. The Doppler shift is known as a change in wavelength that results from hitting a moving target. Doppler Lidar accounts for this shift to analyse the velocity of a subject target.

Because of the versatility of the application, it various uses may include specific applications such as: Mapping of global wind changes in the upper atmosphere to provide a resource for answering questions on global climate.

LIDAR uses the same principle as RADAR (RAdio Detection and Ranging) except that light wavelengths used in LIDAR are 10,000 to 100,000 times shorter than conventional radar. The ability to analyse these shorter wavelengths allows for determination of a broader range of properties beyond just location of hard targets. Generally, once light waves are transmitted from a source the targets they interact with alter them. The light that is reflected back to the source is analysed based on the changed properties of the light to determine a property of the target.

Implementation of LIDAR is being utilized for surface topography. LIDAR has been used in New Orleans for determining levee heights without doing ground survey. Typical LIDAR uses infrared rays that are not effective in retrieving data below water surfaces and has an alleged accuracy of six inches for surface topography.

Shorter frequency blue-green rays now being used for LIDAR allow for water penetration and for bathymetric measurements. This procedure has been used by the United States Army Corps of Engineers in their task of mapping navigable waterways.

Semas suggested a LIDAR sweep of the City's stormwater management facilities to be followed by another sweep several years later. This would allow for a basis of calculation for changes in the bottom elevation of the stormwater management facilities being studied that can be concluded to be accumulated sediment. Advantages and disadvantages of this application are documented below:

Advantages	Disadvantages:
Complete topographic information compiled	Cost may be prohibitive if not done on
Faster than ground survey	widescale basis
Requires no Landowner interface	Not quite as accurate as ground surveys

Although there are LIDAR providers in Canada, the providers specialize in infrared services that will not permit effective bathymetric measurements. However it has been noted that there may be providers available in the United States however the costs of the blue-green technology will be notably higher at this time based on its limited use.

5.0 CONCLUSION

5.1 CONCLUSIONS

The objective of the monitoring study was to ensure that the SWM facilities operate effectively and efficiently. In addition to the Performance Monitoring, it was desired to investigate the condition of the SWM facilities in order to:

- Establish a monitoring program
- Establish a maintenance schedule with associated costs
- Estimate the accumulated sediment

- During the monitoring period, the facility experience no outflow as it has not yet reached the design permanent water level

- Based on field measurements, there is a depth of less than 0.10m (on average) at the deepest area of the sediment forebay. This equates to an estimated accumulated sediment volume within the forebay of 70m^3. This lower than expected volume results from the fact that the tributary area has not been built out to its design condition as yet.

- Sediment loading rates described in the MOE manual account for annual sediment accumulation in a SWM facility that could alter the bottom elevation of a facility anywhere from 0.25m to less than 0.05m. Accordingly, it was felt that a more accurate measure of sediment accumulation rates on a watershed basis could be obtained by surveying SWM facility bathymetry over a course of several years.

- Research for alternative survey methods for SWM Facility bathymetric measurements resulted in an investigation into a Light Detection and Ranging (LIDAR) technology. Based on the results of our investigation it seems the technology may be satisfactory for a study of this nature but there is no availability in Canada for mainstream use.

- No erosion or sedimentation was found in the downstream channel. This is based on the fact that the facility has not reached its design permanent water level as yet and thus rarely experiences outflow. As outlined in the original proposal, the stakes will be left in place at the City's request for future erosion monitoring.

5.2 RECOMMENDATIONS

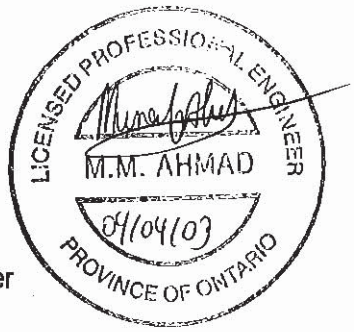
- Site inspection personnel should visually monitor the operations of the quality pond and the control structure every 6 months. The Checklist attached in Appendix B is to be used by the Inspector to itemize any areas of concern.

- Since accumulated sediment volumes are considerably low it is recommended that accumulated sediment be re-examined in 5 years or when the tributary area has been built out closer to its design condition, whichever comes first. At this point it can be determined when maintenance would be required.

- It is estimated that a cost of $98,000 (Cdn. 2003) would be incurred to perform maintenance activity including removal and disposal of accumulated sediment for the SWM Facility No. 4402B – Fletcher's Creek Business Park.

Respectfully submitted,

SERNAS ASSOCIATES



Muneef Ahmad, P.Eng.
Water Resources Engineer

FOR

Ken Chow, P.Eng.
Principal, Manager – Water Resources

APPENDIX G
FIRE FLOW TEST

FIRE FLOW AT 20 PSI CALCULATION

From fire flow test, fill in ***info:*** Applied Fire

Static Pressure, Ps: 87 PSI

Test No.	# of Nozzles	Nozzle Diameter (inches)	Discharge Coeff.	Residual Pressure, Pr (PSI)	Pitot Pressure, Pp (PSI)	Discharge, Qr (US GPM)
1	1	2.5	0.9	86	58	1591
2	2	2.5	0.9	82	36	2601

To calculate flow @ 20 PSI 20 PSI

Qf = Qr x {(Ps – 20)/(Ps – Pr)} ^0.54

Where, Qf = Fire flow in gpm at 20 psi

Qr = Actual flow in gpm	**1591**
Ps = Static pressure	**87**
Pr = Residual pressure	**86**

Qf = **15408** Based on test No. 1

Qf = **11819** Based on test No. 2

Summary Chart

Flow Rat (US GPM)	Flow Rate (L/s)	Flow Rate (L/m)	Head (PSI)
0	0.0	0	87
1591	100.4	6023	86
2601	164.1	9846	82
11819	745.7	**44740**	**20**