FUNCTIONAL SERVICING REPORT

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

CITY OF MISSISSAUGA
REGIONAL MUNICIPALITY OF PEEL

FILE No. 224-M62

DECEMBER 19, 2024

SKIRA & ASSOCIATES LTD.
CONSULTING ENGINEERS

3464 Semenyk Court, Suite 100
Mississauga, Ontario L5C 4P8
(905) 276-5100
info@skiraconsult.ca

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

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. 17.59 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 Figure 2 *for the revised* ***Draft Plan*** *prepared by Design Plan Services Inc. in January 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 – January 2025
- De Zen Fletcher's Creel Hazard Assessment by Parish Geomorphic – August 2011
- A Soil Investigation for Proposed Commercial Development by Soil Engineers Ltd. – July 2008
- 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 – August 2012

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 January 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 January 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.

A large portion of the site will remain undeveloped (5.98 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 January 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. – July 2008), 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 5.5m below the ground surface or at an elevation of 198.30m. For the purposes of hydrologic analysis, the soil was classified as hydrologic soil Group BC.

The Soil Investigation Report and construction recommendations can be found in ***Appendix A***. As the Phase 1 and Phase 2 ESAs, included in *Appendix A*, were completed more than five years ago, updates of these ESAs will be performed per City of Mississauga comments and will be included in future submissions.

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 includes a Limited Phase 2 Environmental Site Assessment for the De Zen Industrial Lands completed by Soil Engineers Ltd. on March 5, 2010. 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 3.1m and 3.4m relative to the existing ground elevations.

The results of their analysis showed 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 Supplementary Slope Stability Study Letter Report from January 21, 2016. 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. Catchments 3 and 4 drain directly to Fletchers Creek. External Catchments Ext.5 through Ext.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 conservative measure.

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.

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
1	2.10	0.25	0.31	0.16	77	89	8	0.84	0.32
2	1.19	0.25	0.31	0.70	77	89	8	2.41	0.10

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 2.00m^3/s and is generated by the 100-yr SCS 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 (m^3/s)
100-yr 12-hr SCS	2.00
100-yr 4-hr Chicago	1.31
Regional (Hurricane Hazel)	1.00

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 of 12.491 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 (3.85 Ha) and Phase 2 (7.77Ha) 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. As the City of Mississauga does not have a standard 30m right-of-way cul-de-sac, a custom cul-de-sac was designed based on City of Mississauga Standard No. 2211.250. Dimensions of this cul-de-sac are included in ***Figures 4 & 5***.

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, 2019) 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

Erosion control for Phase 2 will include an erosion control storage tank size to control the post-development 25mm flow to pre-development levels. A Rational Method Model was used for the preliminary sizing of the erosion control volume shown on ***Figure 5*** and ***Figure*** *7*. The pre-development discharge of the Phase 2 lands was determined to be $0.044m^3/s$ as shown in ***Appendix C***. This flow rate was used as the discharge rate to calculate volume required for the erosion control storage tank. The storage required to maintain this flow rate under post-development conditions was determined to be $929.49m^3$.

This volume will be provided with the cultec chambers located in front of each building. A combined volume of $100m^3$ will be provided in two (2) locations as shown on ***Figure 5***. Infiltration of roof water discharge is considered to ensure only “clean runoff” is being infiltrated.

YEAR STORM
25mm EROSION
CITY
MISSISSAUGA

C = 0.900
A (ha) = 6.20000
Allow. Discharge Qa (m³/s) = 0.03500
Safety Factor Sf = 0%

Max. Required Detention (m³) = ***934.66***

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	1.2109	363.28	10.50	352.78
10	56.33	0.8732	523.91	21.00	502.91
15	44.77	0.6940	624.57	31.50	593.07
20	37.50	0.5813	697.51	42.00	655.51
25	32.46	0.5031	754.71	52.50	702.21
30	28.74	0.4455	801.84	63.00	738.84
35	25.87	0.4010	842.00	73.50	768.50
40	23.58	0.3654	877.05	84.00	793.05
45	21.70	0.3364	908.21	94.50	813.71
50	20.14	0.3121	936.28	105.00	831.28
55	18.80	0.2915	961.87	115.50	846.37
60	17.66	0.2737	985.41	126.00	859.41
65	16.66	0.2583	1007.22	136.50	870.72
70	15.78	0.2447	1027.57	147.00	880.57
75	15.01	0.2326	1046.64	157.50	889.14
80	14.31	0.2218	1064.61	168.00	896.61
85	13.68	0.2121	1081.60	178.50	903.10
90	13.12	0.2033	1097.73	189.00	908.73
95	12.60	0.1953	1113.09	199.50	913.59
100	12.13	0.1880	1127.75	210.00	917.75
105	11.69	0.1812	1141.79	220.50	921.29
110	11.29	0.1750	1155.25	231.00	924.25
115	10.92	0.1693	1168.19	241.50	926.69
120	10.58	0.1640	1180.66	252.00	928.66
125	10.26	0.1590	1192.69	262.50	930.19
130	9.96	0.1544	1204.30	273.00	931.30
135	9.68	0.1501	1215.54	283.50	932.04
140	9.42	0.1460	1226.44	291.90	934.53
145	9.17	0.1422	1237.00	302.34	**934.66**
150	8.94	0.1386	1247.26	312.77	934.49
155	8.72	0.1352	1257.23	323.20	934.03

6.3 Potential Additional LID Measures

In addition to the cultic bottomless trench, LID BMPs 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 attenuation/retention and filtration within the native material. The potential LID measures will be designed to provide a minimum of 5mm runoff retention where feasible, to provide attenuation, 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, April 2008). 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, 3.85 Ha required 5mm runoff to be retained on site is as follows:

$$V_{5mm} = 38{,}500 \times 0.005$$
$$= 192.50 m^3$$

Parking areas in front of Buildings B & C will be constructed as permeable surface. Approx. 2,100m³ of permeable paving is suggested.

$V_{PROVIDED}$ = 2,100 x 0.13 x 0.4
= 252m³

Where, 0.30m represents typical pavement thickness.

6.4.2 Phase 2 – 5mm Water Balance

Using impervious areas of this portion of the development approx. 6.2 Ha representing 80% coverage typical for industrial buildings. Required 5mm runoff to be retained on site is as follows:

V_{5mm} = 62,000 x 0.005
= 310m³

Cultec system based gravel layer provides approx. 168m³ of the required storage.

$V_{PROVIDED}$ = 1,400 x 0.3 x 0.4
= 168m³

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

Approx. 1,200m², located at visitor parking area, will provide required storage volume.

$V_{PROVIDED}$ = 1,200 x 0.3 x 0.4
= 144m³

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

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 C*** *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.

$Q_{2\text{-}yr}$ = 7.77 x 0.25 x 59.89 / 360
= $0.323 m^3/s$

$Q_{100\text{-}yr}$ = 7.77 x 0.25 x 140.69 / 360
= $0.759 m^3/s$

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 (m^3/s) = 0.323000
Safety Factor Sf = 0%

Max. Required Detention (m^3) = 2948.90

RAINFALL DURATION *Tc (min)*	RAINFALL.L INTENSITY *I (mm/hr)*	TOTAL UNCONTROLLED RUNOFF *Q=CIA/360 (m^3/sec)*	INFLOW VOLUME Vi (m^3)	OUTFLOW VOLUME Vo (m3)	REQUIRED DETENTION VOLUME (m^3) *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	201.70	202.20	0.50	1,409
Building B	201.70	202.20	0.50	755
Building C	201.70	202.20	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 **0.323m^3/s** will be achieved by the means of an orifice restrictor tube installed over the outlet pipe at STMMH 1 located at the property line southwest corner. The size of the orifice restrictor pipe is **385mm dia.**

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 D** 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

$$\text{Peak Factor} = 1 + \frac{14}{4 + 0.303^{0.5}}$$
$$= 1 + 3.07$$
$$= 4.07 \simeq \text{(max. 4.0 factor)}$$

$$\text{Expected Peak Flow Rate} = 302.8 \times 303 \times 4.0$$
$$= 366{,}993.6 \text{ L/day} = 4.25 \text{ L/s}$$

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 – 7.77 x 70p/hectares = 544 population

Peak Factor $= 1 + \dfrac{14}{4 + 0.544^{0.5}}$

$= 1 + 2.95$

$= 3.95$

Expected Peak Flow Rate = 302.8 x 544 x 3.95
= 650,656.6 L/day = 7.53 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 will be performed and watermain pressures will be determined as soon as weather permits.

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 1999, 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 = 12,570
F = Fire Flow in L/m

$F = 220 \times 0.60 \times \sqrt{12{,}570}$
$= 14{,}800$ L/min

Calculated value can be reduced by 50% if automatic sprinkler system is provided. Therefore,

$F = 14{,}800 \times 0.50$
$= 74{,}000$ L/min

Further reduction can be applied to the fire flow demands when the content and level of hazard protection of the industrial building is known.

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 principal the recommendation.

Yours truly,

SKIRA & ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
M. JOZWIK
Dec. 19, 2024
PROVINCE OF ONTARIO

Michael Jozwik, P. Eng.
MJ:ak

NOTE: ***<u>Limitation of Report</u>***

*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

Phase 2 ESA & Slope Stability Letter
By Soil Engineers Ltd.

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

BARRIE	MISSISSAUGA	OSHAWA	NEWMARKET	GRAVENHURST	HAMILTON
TEL: (705) 721-7863	TEL: (905) 542-7605	TEL: (905) 440-2040	TEL: (905) 853-0647	TEL: (705) 684-4242	TEL: (905) 777-7956
FAX: (705) 721-7864	FAX: (905) 542-2769	FAX: (905) 725-1315	FAX: (905) 881-8335	FAX: (705) 684-8522	FAX: (905) 542-2769

April 13, 2020 Reference No. 0803-S002
(Revision of Report dated January 21, 2016)

678604 Ontario Inc.
c/o James Lethbridge Planning Inc.
2030 Bristol Circle, Unit 201
Oakville, Ontario
L6H 0H2

Attention: Mr. James Lethbridge

Re: Supplementary Slope Stability Study Letter Report
Proposed Employment Lands
Dezen Industrial – Phase 2
Southwest Quadrant of Highway 407 and Hurontario Street
City of Mississauga

Dear Sir:

Further to the email request from Mr. James Lethbridge dated December 15, 2015, we have reviewed the comments issued by the Credit Valley Conservation (CVC) for the captioned site. Furthermore, we have received the toe erosion allowance and meander belt setbacks calculated and presented by GEO Morphix Ltd. In their Tributary of Fletcher's Creek Erosion Hazard Assessment. In response, we herein present our supplementary slope stability study findings and recommendations, incorporating the updated setbacks.

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 report, an addendum was issued in 2012 to provide additional analyses and clarifications to address the 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.

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 well data, the use of this 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 provided by James Lethbridge Planninc Inc.; 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 force-moment-equilibrium criteria of the Bishop Method with the 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 shows that the existing slope has a FOS of 1.45, which fails to meet the OMNR requirements. The result is presented on Drawing No. 2. Therefore, the existing valley slope at this location is considered to be geotechnically unacceptable for the proposed development. A gradient of 1V:2.2H is recommended for use in 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 toe erosion allowance 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 toe erosion setback. 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 toe erosion allowance (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). Where the valley feature is not well defined east/opposite of Cross-Section E-E, the LTSSL follows the meander belt setback.

Lastly, a development setback buffer for man-made and environmental degradation of the bank will be required. The distance of the buffer is 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:

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.

1. 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.
2. 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.
3. 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, B.A.Sc.

Bernard Lee, P.Eng.

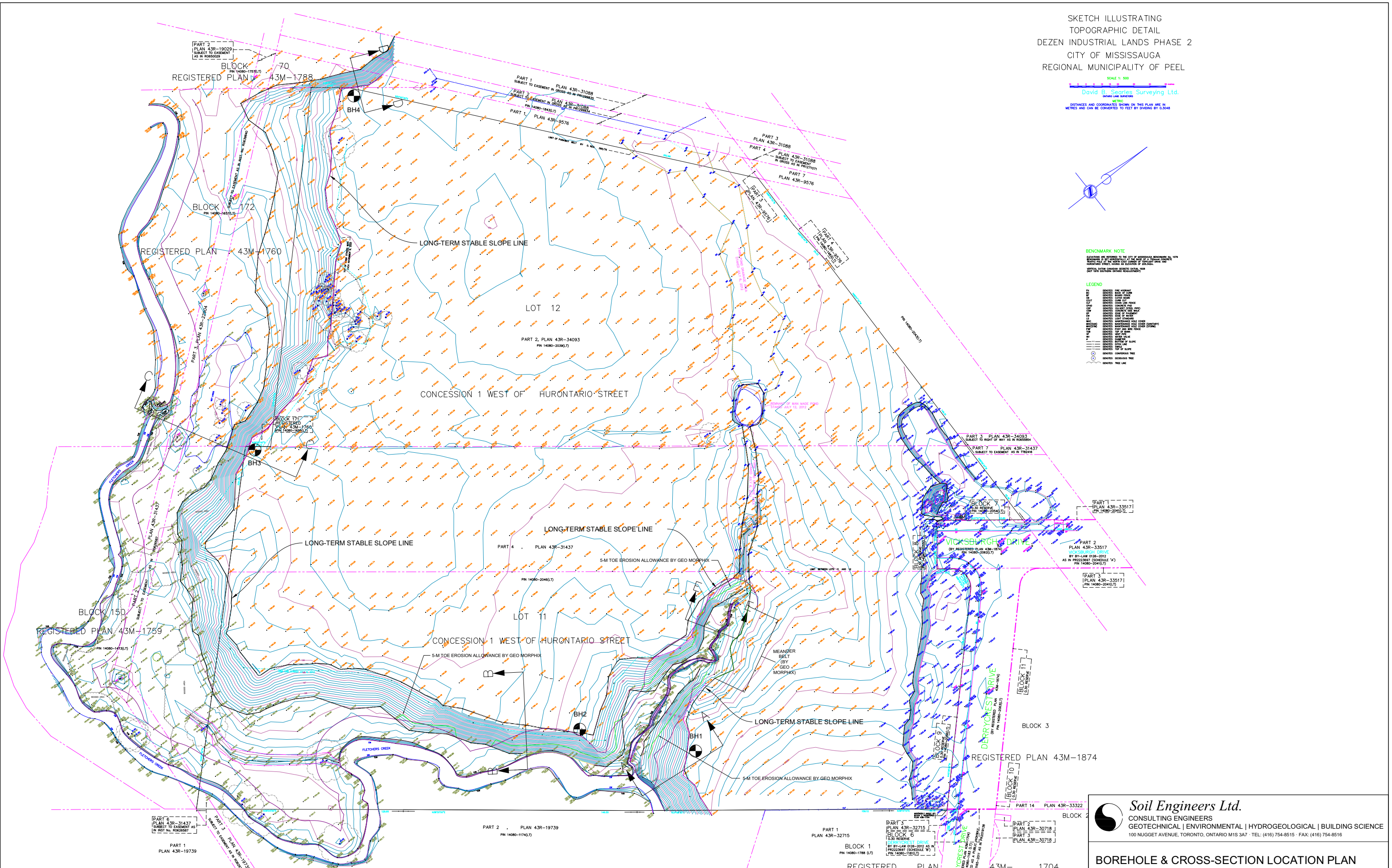
LICENSED PROFESSIONAL ENGINEER
Apr 13, 2020
B. P. Y. LEE
100104568
PROVINCE OF ONTARIO

HWY/BL

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

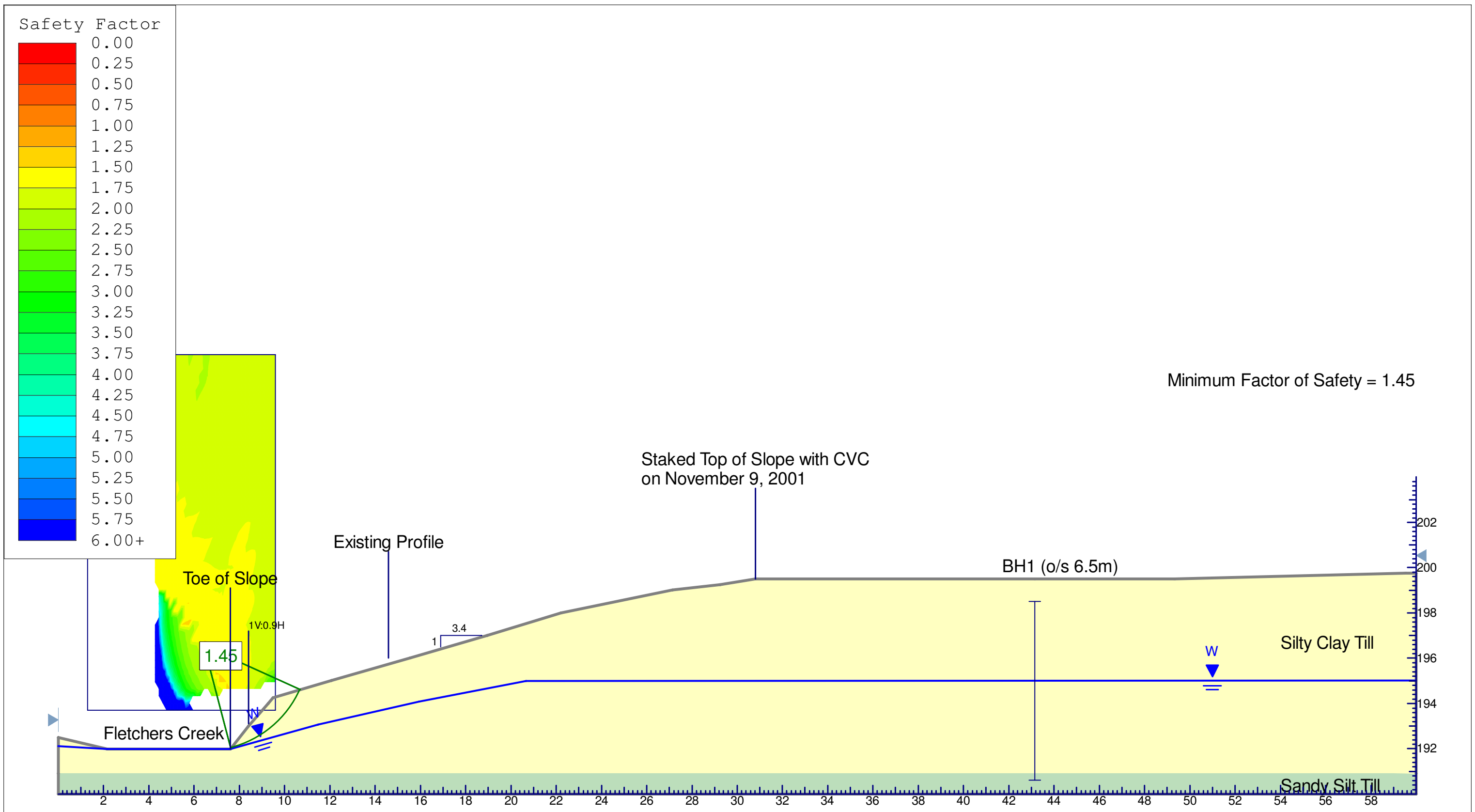
c. Soil Engineers Ltd. (Mississauga)
Attn: Mr. Benjamin Lee, Branch Manager



Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 · TEL: (416) 754-8515 · FAX: (416) 754-8516

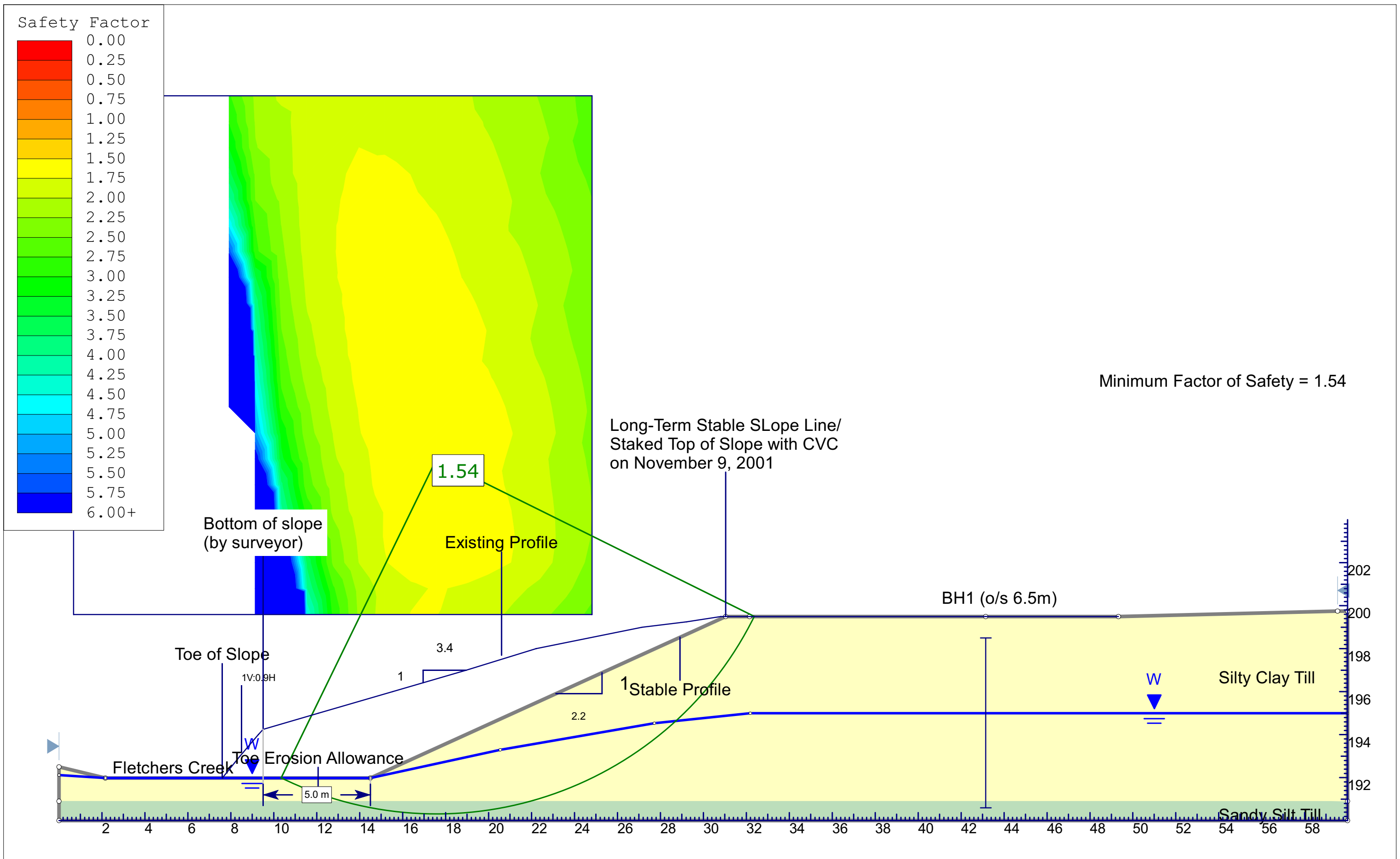
BOREHOLE & CROSS-SECTION LOCATION PLAN

SITE	DEZEN INDUSTRIAL - PHASE 2 SW OF HIGHWAY 407 & HURONTARIO STREET, MISSISSAUGA		
DESIGNED BY	CHECKED BY	DWG NO. 1	
SCALE 1:1000	REF. NO. 0803-S002	DATE APRIL 2020	REV C

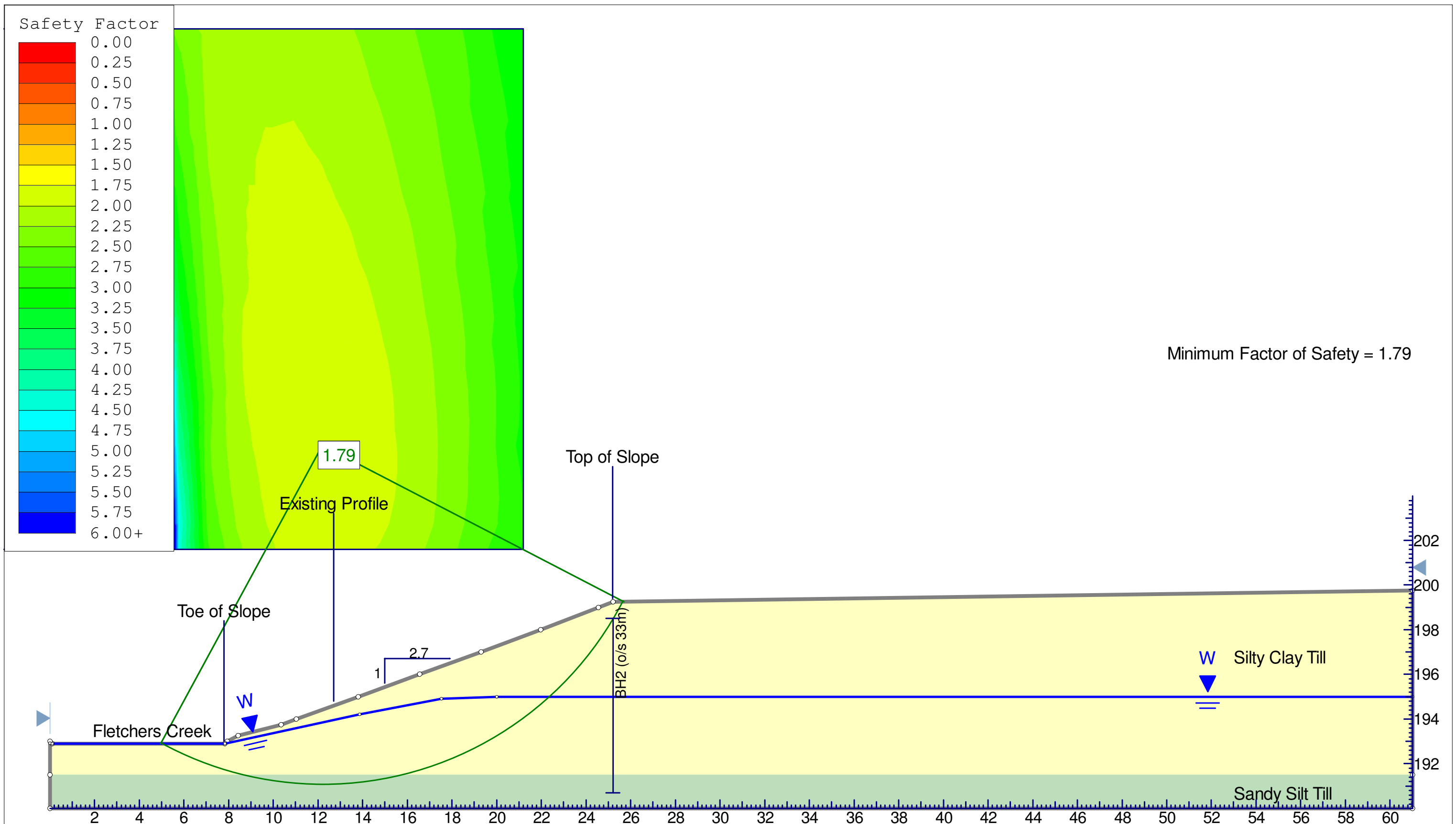


Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 · TEL: (416) 754-8515 · FAX: (416) 754-8516

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

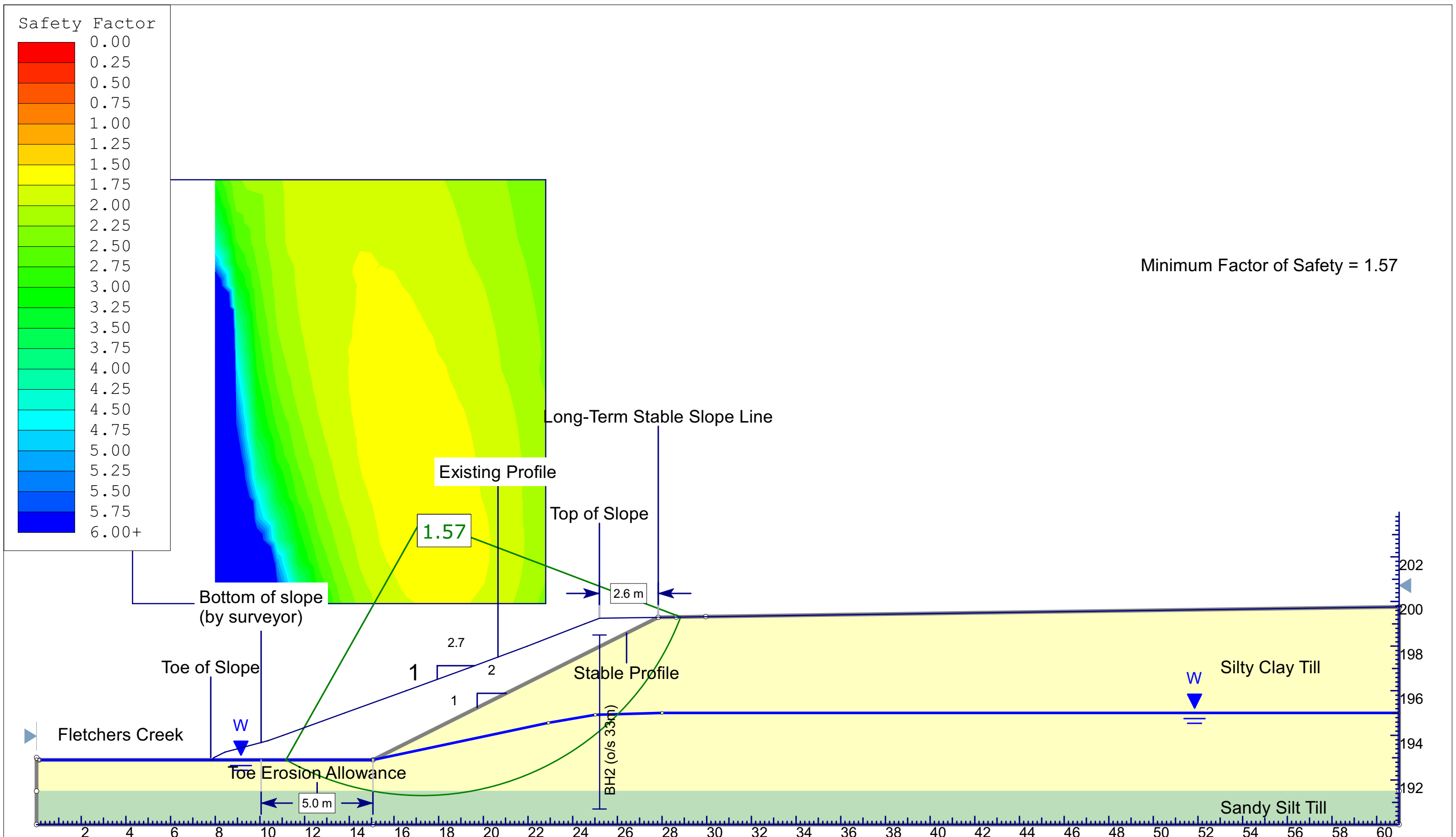


Soil Engineers Ltd. CONSULTING ENGINEERS GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE 90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335	Project Title	Cross-Section A-A			Load Case	Stable Condition
	Location	Southwest 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



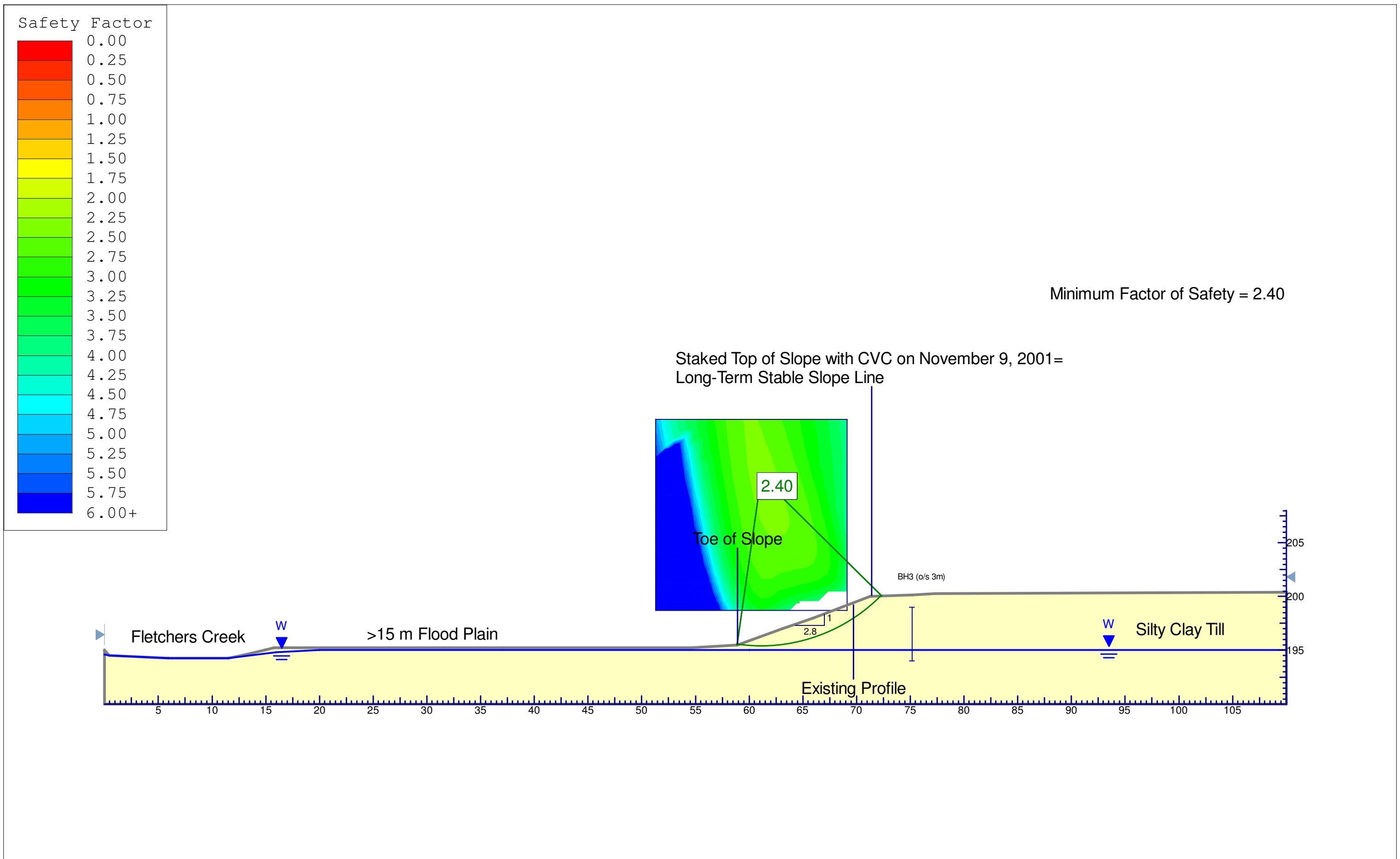
Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 · TEL: (416) 754-8515 · FAX: (416) 754-8516

Project Title		Load Case	
Cross-Section B-B		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



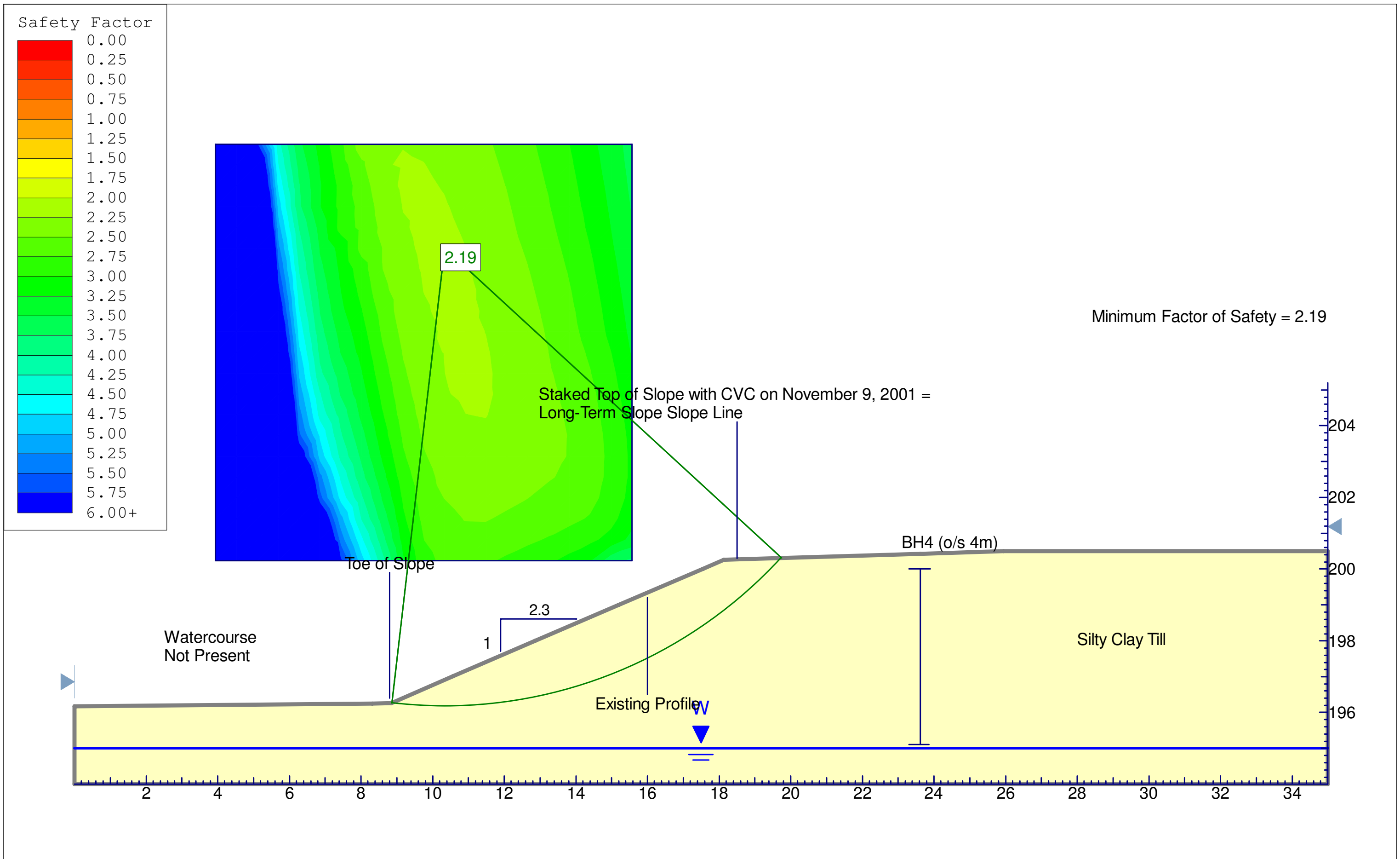
Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
90 WEST BEAVER CREEK ROAD, SUITE #100, RICHMOND HILL, ONTARIO L4B 1E7 · TEL: (416) 754-8515 · FAX: (905) 881-8335

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

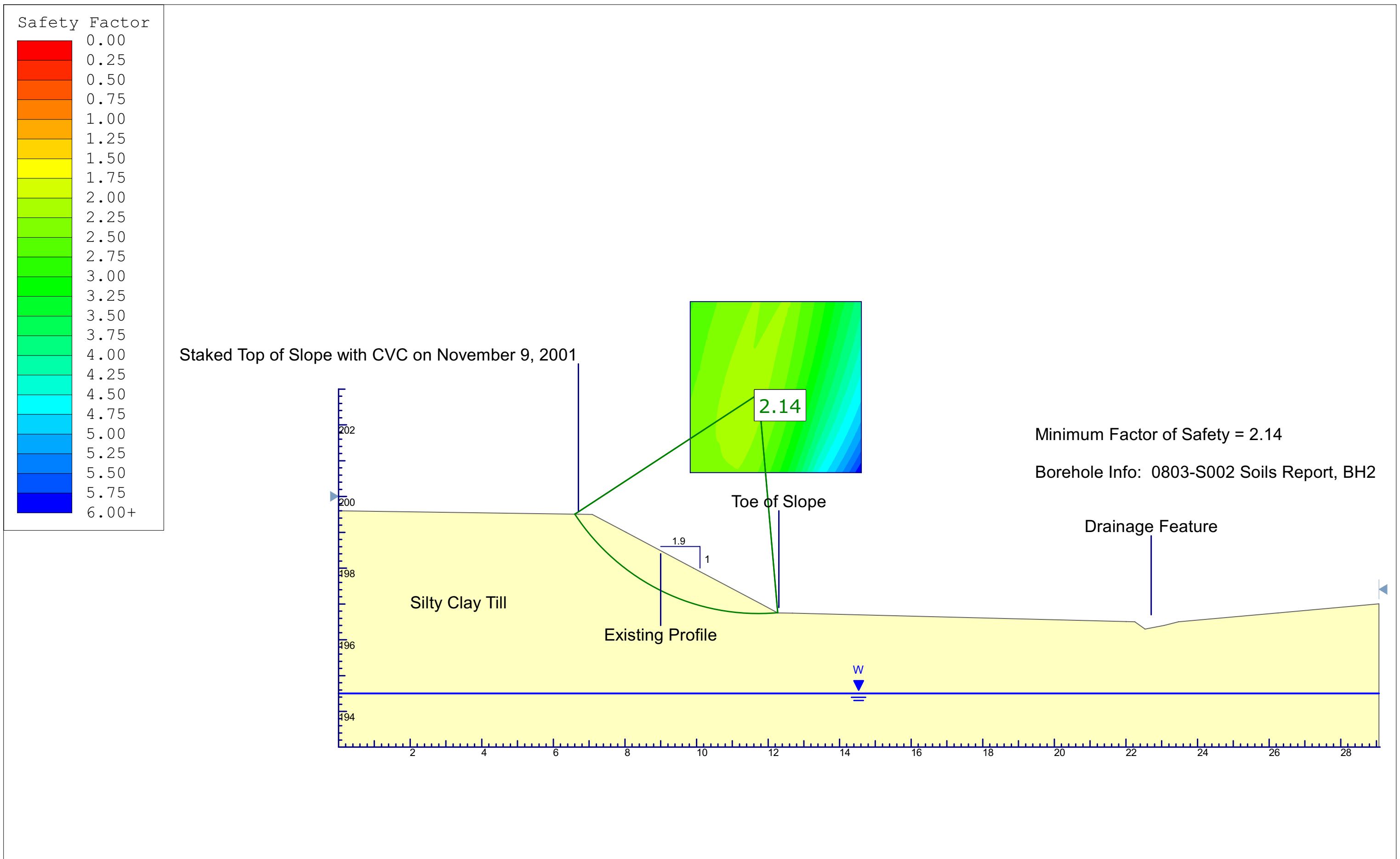


Soil Engineers Ltd.
CONSULTING ENGINEERS
GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE
100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 · TEL: (416) 754-8515 · FAX: (416) 754-8516

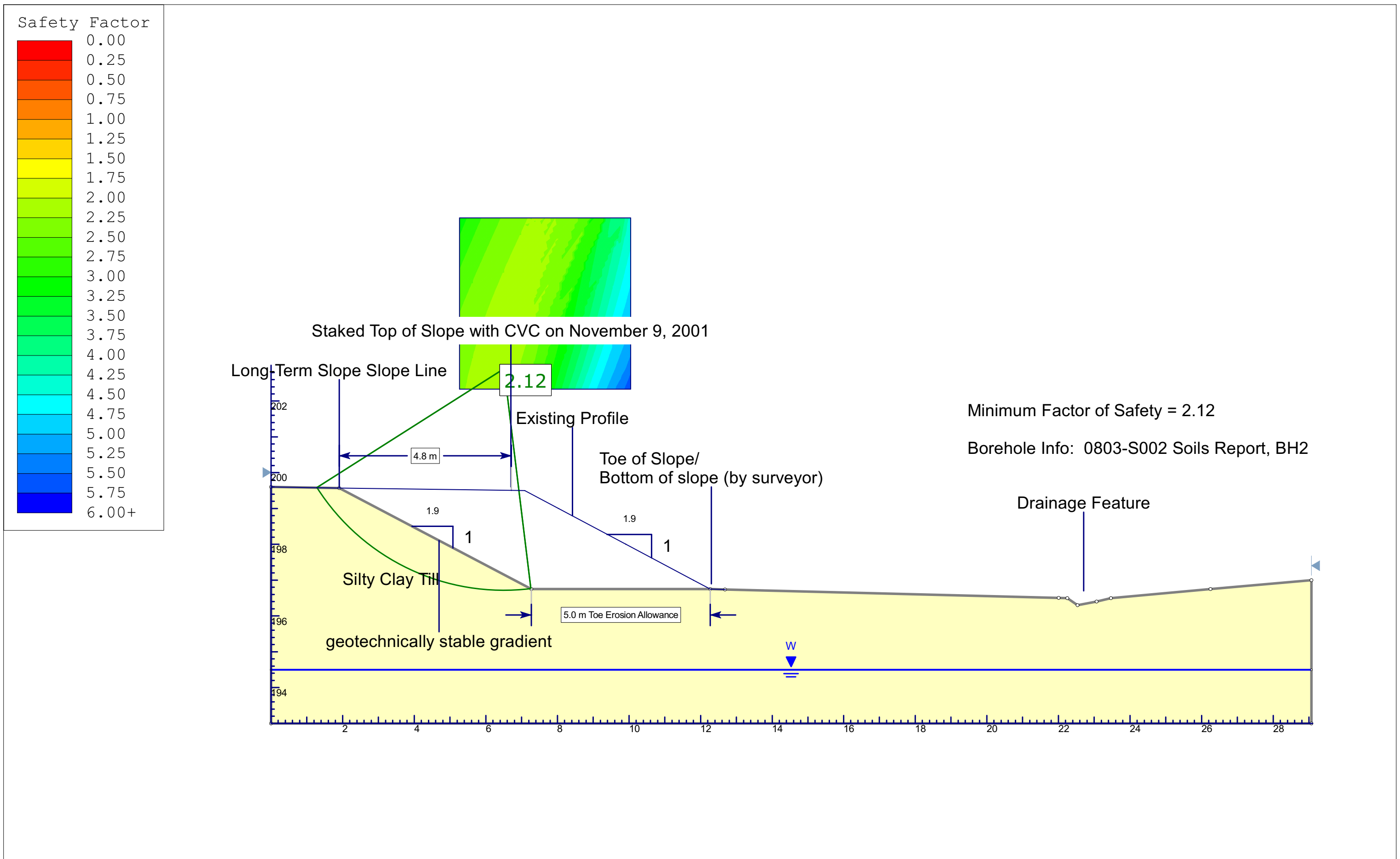
Project Title: Cross-Section C-C			Load Case: Existing Condition
Location: Southwest Quadrant of Highway 407 and Hurontario Street, Mississauga			
Drawn By: BL	Checked By:	Scale: 1:500	Revision: -
Date: January 2016		Reference No.: 0803-S002	Drawing No.: 6



Soil Engineers Ltd. CONSULTING ENGINEERS GEOTECHNICAL | ENVIRONMENTAL | HYDROGEOLOGICAL | BUILDING SCIENCE 100 NUGGET AVENUE, TORONTO, ONTARIO M1S 3A7 · TEL: (416) 754-8515 · FAX: (416) 754-8516	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



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	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



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

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

APPENDIX B

Stormwater Design Sheets
Pre-Development Sub-Catchment Summary
External Flow Visual OTTHYMO Results
Culvert Sizing – Culvert Master Calculations
Cultec System – Erosion Control Surface Volume Calculations
Quality Control Oil/Grit Separator Calculations

SUBDIVISION : **DEZEN CONSTRUCTION COMPANY LTD**

CITY OF MISSISSAUGA

SHEET No. 1 of 1

MAJOR DRAINAGE **FLETCHER'S CREEK**

CITY FILE:

CONSULTANT : **SKIRA & ASSOCIATES LTD.**

STORM SEWER DESIGN CHART

FINAL SUBMISSION

PROJECT No. : 220-M10

DESIGNED BY : M.J.

DATE : APR. 2020

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

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	7.66	0.90	6.89	7.66	6.89	15.00	99.17	**1.898**	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                         *
*                          PER DEVELOPMENT                                  *
*                  STORMWATER MANAGEMENT PRACTICES                          *
*                                                                           *
*****************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                               *
*  100 YEAR DESIGN STORM - 4 HR CHICAGO DISTRIBUTION                        *
*****************************************************************************

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=3.29 HA DWF=0 CN=77 TP=0.10
                       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
                       SCS24.100
*
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

Distributed by the INTERHYMO Centre. Copyright (c), 1989.  Paul Wisner & Assoc.
  EXCLUSIVE USE TO : PAUL WISNER AND ASSOCIATES I

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



DATE: 04-27-2000                      TIME: 14:11:47

COMMENTS: ____________________________________________________________


-------------------------------------------------------------------------------
******************************************************************************
*                                                                            *
*                     SKIRA & ASSOCIATES LIMITED                             *
*                           MARCH   2020                                     *
*                    DEZEN INDUSTRIAL   SUBDIVISION                          *
*                         PER DEVELOPMENT                                    *
*                 STORMWATER MANAGEMENT PRACTICES                            *
*                                                                            *
******************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                                *
*  100 YEAR DESIGN STORM - 4 HR CHICAGO DISTRIBUTION                         *
******************************************************************************
  ****************************
  ** SIMULATION NUMBER:   1 **
  ****************************


--------------------
|    READ STORM    |    Filename: 100MIS4.CHI
| Ptotal= 79.43 mm |    Comments: CHICAGO - 100 YR STORM DISTRIBUTION - MI
--------------------
                 TIME    RAIN |  TIME    RAIN |  TIME    RAIN |  TIME    RAIN
                  hrs   mm/hr |   hrs   mm/hr |   hrs   mm/hr |   hrs   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)=   3.29   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)=     .10

     Unit Hyd Qpeak  (cms)=   1.257

     PEAK FLOW       (cms)=    .603 (i)
     TIME TO PEAK    (hrs)=   1.417
     RUNOFF VOLUME    (mm)=  38.460
     TOTAL RAINFALL   (mm)=  79.430
     RUNOFF COEFFICIENT   =    .484

     (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):      3.29      .60     1.42    38.46
          ================================================
          ID = 9 (0114):     15.35     1.31     1.50    36.16

     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                           *
*                         PER DEVELOPMENT                                     *
*                 STORMWATER MANAGEMENT PRACTICES                             *
*                                                                             *
*******************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                                 *
*  100 YEAR DESIGN STORM - 4 HR CHICAGO DISTRIBUTION                          *
*******************************************************************************

--------------------
|    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) |
|   5 +  6 =  7    |         AREA    QPEAK    TPEAK     R.V.
--------------------         (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) |   Area    (ha)=   3.29   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)=    .10

     Unit Hyd Qpeak  (cms)=   1.257

     PEAK FLOW       (cms)=    .244 (i)
     TIME TO PEAK    (hrs)=   5.250
     RUNOFF VOLUME    (mm)=  45.295
     TOTAL RAINFALL   (mm)=  88.540
     RUNOFF COEFFICIENT   =    .512

     (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      .71     5.33    42.02
        + ID2= 8 (0108):     3.29      .24     5.25    45.30
          ===============================================
          ID = 9 (0114):    15.35      .95     5.25    42.72

     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                           *
*                         PER DEVELOPMENT                                     *
*                 STORMWATER MANAGEMENT PRACTICES                             *
*                                                                             *
*******************************************************************************
*  PRE-DEVELOPMENT CONDITIONS                                                 *
*  100 YEAR DESIGN STORM - 4 HR CHICAGO DISTRIBUTION                          *
*******************************************************************************

--------------------
|    READ STORM    |    Filename: SCS24.100
| Ptotal=138.73 mm |    Comments: SCS 24 HR TYPE II - 100 YR STORM DISTRIB
--------------------
                 TIME    RAIN | TIME    RAIN | TIME    RAIN | TIME    RAIN
                  hrs   mm/hr |  hrs   mm/hr |  hrs   mm/hr |  hrs   mm/hr
                  .25     .77 | 6.25    2.23 | 12.25  62.92 | 18.25   2.51
                  .50    1.53 | 6.50    2.78 | 12.50  20.04 | 18.50   2.51
                  .75    1.53 | 6.75    2.78 | 12.75  15.17 | 18.75   2.51
                 1.00    1.53 | 7.00    2.78 | 13.00  10.30 | 19.00   2.51
                 1.25    1.53 | 7.25    2.78 | 13.25   6.12 | 19.25   2.51
                 1.50    1.53 | 7.50    2.78 | 13.50   1.95 | 19.50   2.51
                 1.75    1.53 | 7.75    2.78 | 13.75   6.68 | 19.75   2.51
                 2.00    1.53 | 8.00    2.78 | 14.00  11.41 | 20.00   2.51
                 2.25    1.67 | 8.25    2.78 | 14.25   7.80 | 20.25   2.09
                 2.50    1.81 | 8.50    3.76 | 14.50   4.18 | 20.50   1.67
                 2.75    1.81 | 8.75    3.76 | 14.75   4.18 | 20.75   1.67
                 3.00    1.81 | 9.00    3.76 | 15.00   4.18 | 21.00   1.67
                 3.25    1.81 | 9.25    3.76 | 15.25   4.18 | 21.25   1.67
                 3.50    1.81 | 9.50    4.45 | 15.50   4.18 | 21.50   1.67
                 3.75    1.81 | 9.75    4.73 | 15.75   4.18 | 21.75   1.67
                 4.00    1.81 |10.00    5.01 | 16.00   4.18 | 22.00   1.67
                 4.25    2.02 |10.25    5.71 | 16.25   3.34 | 22.25   1.67
                 4.50    2.23 |10.50    6.40 | 16.50   2.51 | 22.50   1.67
                 4.75    2.23 |10.75    7.52 | 16.75   2.51 | 22.75   1.67
                 5.00    2.23 |11.00    8.63 | 17.00   2.51 | 23.00   1.67
                 5.25    2.23 |11.25   11.00 | 17.25   2.51 | 23.25   1.67
                 5.50    2.23 |11.50   13.36 | 17.50   2.51 | 23.50   1.67
                 5.75    2.23 |11.75   59.58 | 17.75   2.51 | 23.75   1.67
                 6.00    2.23 |12.00  105.79 | 18.00   2.51 | 24.00   1.67

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

*
* --------------------------------------------------
*  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)=    .165 (i)
     TIME TO PEAK    (hrs)=  12.167
     RUNOFF VOLUME    (mm)= 103.300
     TOTAL RAINFALL   (mm)= 138.730
     RUNOFF COEFFICIENT   =    .745

     (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
```

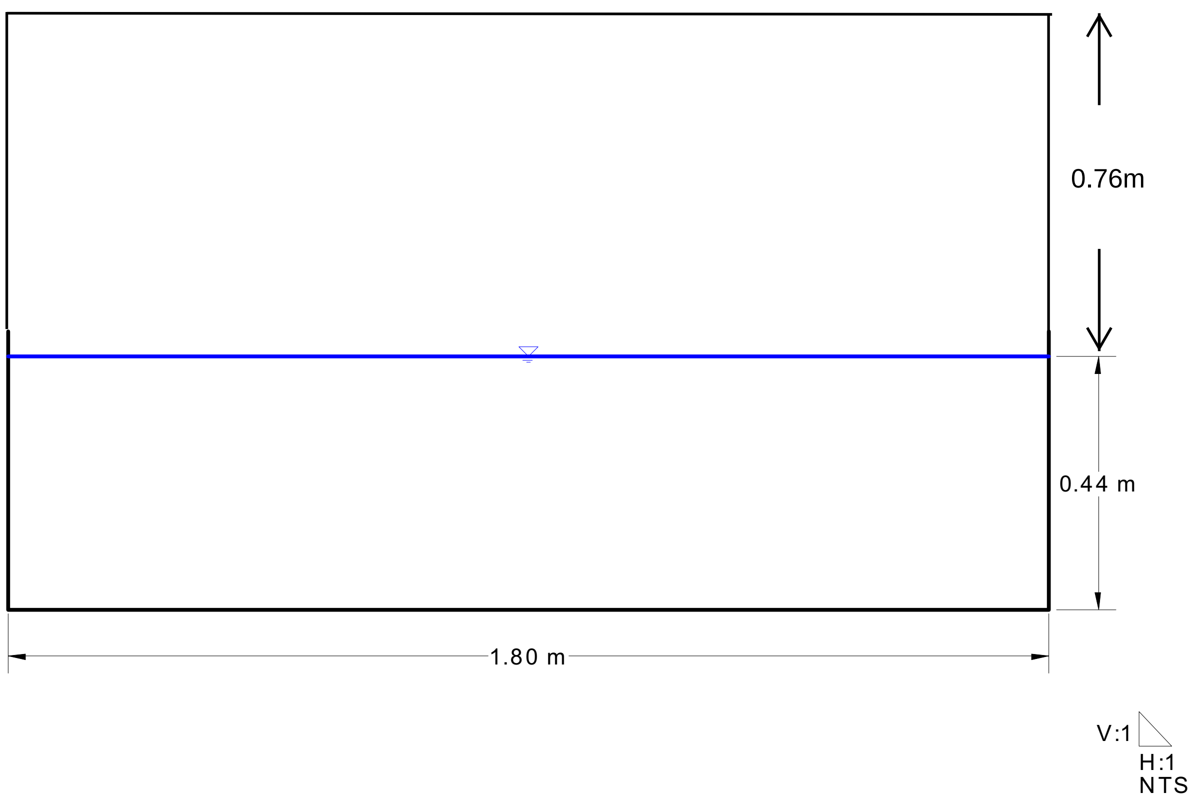
```
     PEAK FLOW       (cms)=    .443 (i)
     TIME TO PEAK    (hrs)=  12.333
     RUNOFF VOLUME    (mm)=  68.711
     TOTAL RAINFALL   (mm)= 138.730
     RUNOFF COEFFICIENT   =    .495

     (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      .16    12.17   103.30
        + ID2= 2 (0102):      4.18      .44    12.33    68.71
          ==================================================
          ID = 3 (0111):      5.06      .60    12.25    74.73

     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)=    .625 (i)
     TIME TO PEAK    (hrs)=  12.250
     RUNOFF VOLUME    (mm)=  84.830
     TOTAL RAINFALL   (mm)= 138.730
     RUNOFF COEFFICIENT   =    .611

     (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      .60    12.25    74.73
        + ID2= 4 (0104):      4.45      .62    12.25    84.83
          ==================================================
          ID = 5 (0112):      9.51     1.22    12.25    79.45

     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)=    .331 (i)
     TIME TO PEAK    (hrs)=  12.333
     RUNOFF VOLUME    (mm)=  88.355
     TOTAL RAINFALL   (mm)= 138.730
     RUNOFF COEFFICIENT   =    .637

     (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     1.22    12.25    79.45
        + ID2= 6 (0106):      2.55      .33    12.33    88.35
          ==================================================
          ID = 7 (0113):     12.06     1.54    12.25    81.34
```

```
     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

--------------------------------------------------------------------------------
*
* ------------------------------------------------------
*  TEMP FLOW FROM EXISTING CATCHMENT AREA TO BE DEVELOPED
* ------------------------------------------------------
*
--------------------
| DESIGN           |
| NASHYD    (0108) |   Area    (ha)=   3.29   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)=    .10

     Unit Hyd Qpeak  (cms)=   1.257

     PEAK FLOW       (cms)=    .650 (i)
     TIME TO PEAK    (hrs)=  12.000
     RUNOFF VOLUME    (mm)=  86.075
     TOTAL RAINFALL   (mm)= 138.730
     RUNOFF COEFFICIENT   =    .620

     (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     1.54    12.25    81.34
        + ID2= 8 (0108):     3.29      .65    12.00    86.07
          ==================================================
          ID = 9 (0114):    15.35     2.00    12.25    82.35

     NOTE:  PEAK FLOWS DO NOT INCLUDE BASEFLOWS IF ANY.

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

 FINISH
================================================================================

→
* ---------------------------------------------
*  RE-RUN for AE
```

Cross Section
Cross Section for Rectangular Channel

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

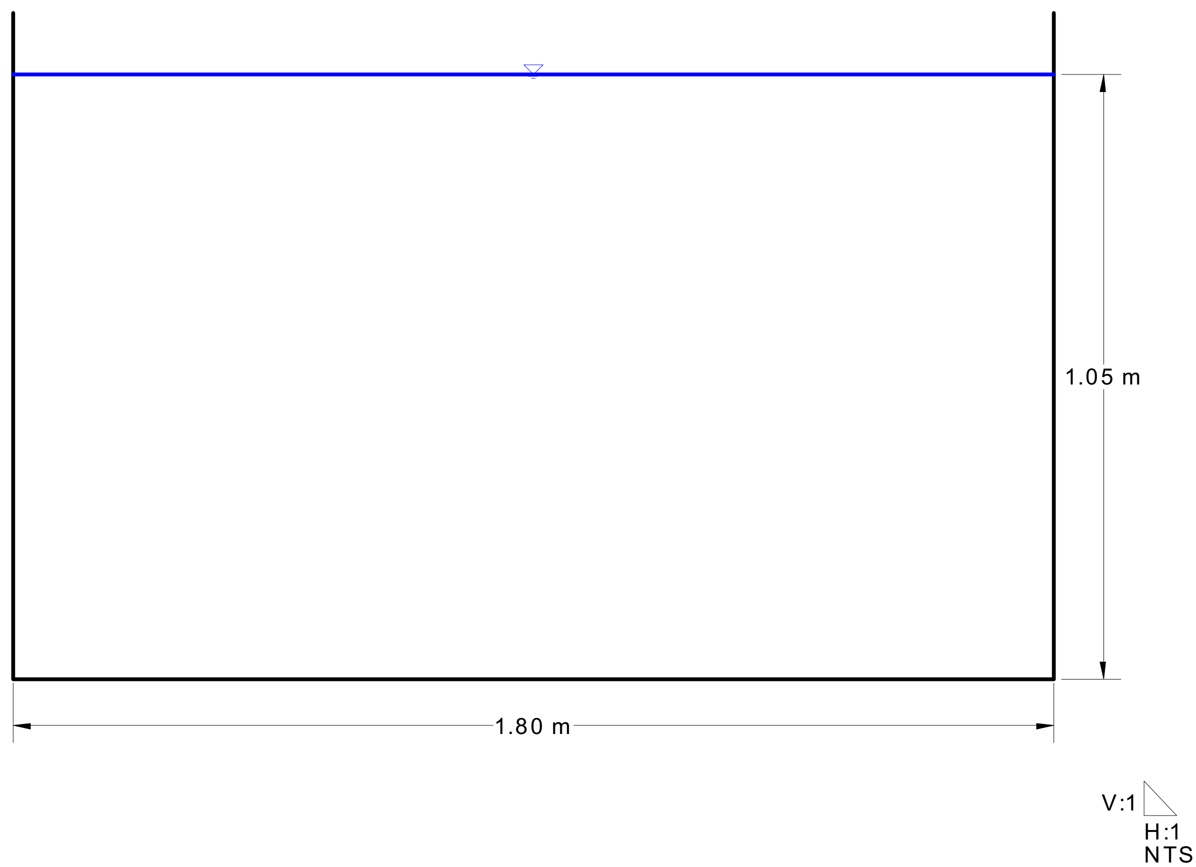
Section Data		
Mannings Coeffic	0.013	
Slope	000500	m/m
Depth	0.44	m
Bottom Width	1.80	m
Discharge	0.6000	m³/s



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	1.05	m
Bottom Width	1.80	m
Discharge	2.0000	m³/s



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	30 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 =		240 pcs	**240.00 pcs**
240 pcs x 7.5' =		1800 LF	**548.64 m**
Number of Rows =		8 rows	**8.00 rows**
Total LF of chambers =		1800 LF	**548.64 m**
1800 ' x 8.933 CF/LF =		**16079.4 CF**	**455.05 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	12.89 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	236.16 feet		71.98 m
Bed depth	3.83 feet		1.17 m
Total CF of effective excavated area	37569.1 CF		1063.21 CM
Total min. excavated area	11451.07 CF		324.07 CM
Total CF volume of HVLV Header & Recharger Chambers =		16734.01 CF	473.57 CM
Total stone required =	20835.11 CF 771.67 CY		589.63 CM
Storage provided within stone =	8334.044 CF		235.85 CM

Total storage within CULTEC Stormwater System = **25068.05 CF** / **709.43 CM**

MATERIALS LIST

MODEL	QUANTITY
Recharger V8 IHD Intermediate Heavy Duty	240
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
PH: 203-775-4416
FX: 203-775-1462
www.cultec.com
custservice@cultec.com

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.

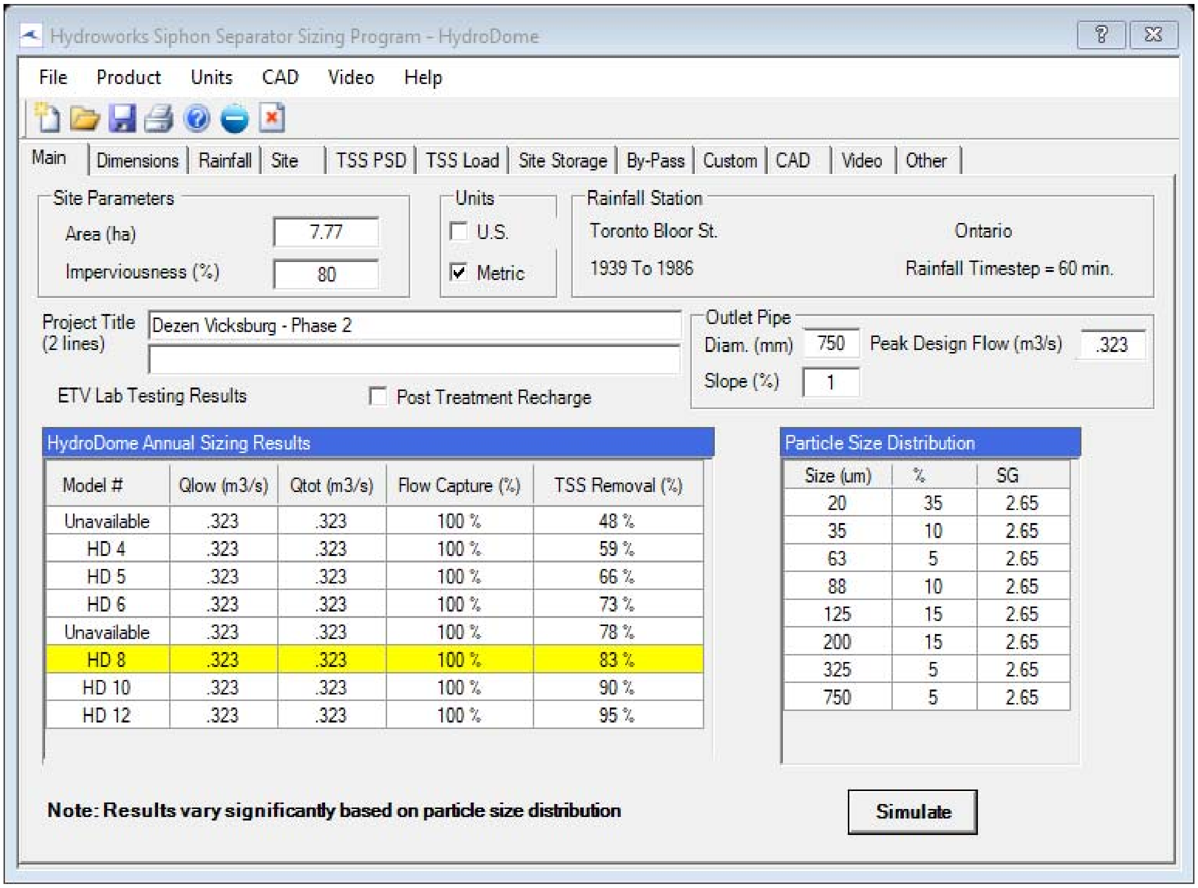
The HydroDome has a siphon which creates a discontinuity in headloss. The given peak flow of .323 (m3/s) Is less than the full pipe flow of 1.11 (m3/s) indicating free flow in the pipe during the peak flow assuming no tailwater condition. Partial pipe flow was assumed for the headloss calculations. The 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 .

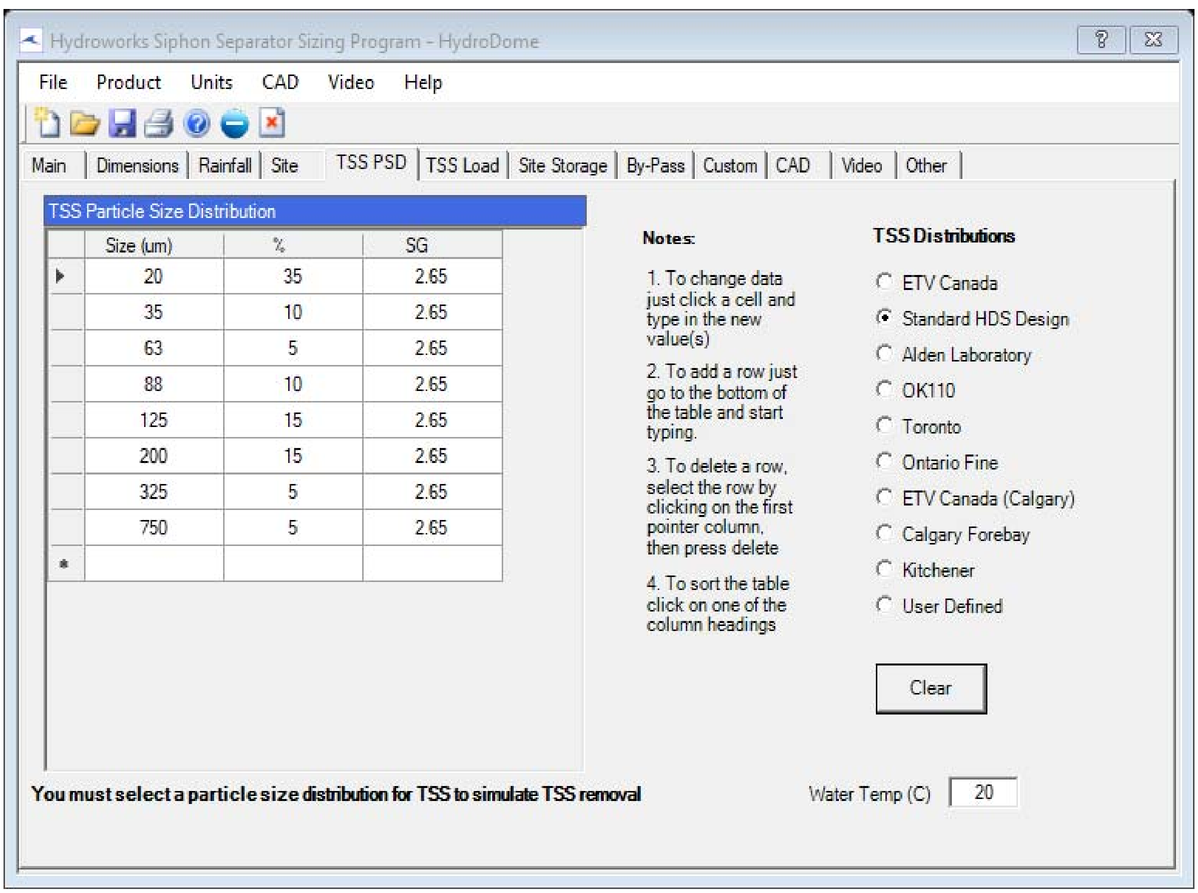
TSS Removal Sizing Summary



Model #	Qlow (m3/s)	Qtot (m3/s)	Flow Capture (%)	TSS Removal (%)
Unavailable	.323	.323	100 %	48 %
HD 4	.323	.323	100 %	59 %
HD 5	.323	.323	100 %	66 %
HD 6	.323	.323	100 %	73 %
Unavailable	.323	.323	100 %	78 %
HD 8	.323	.323	100 %	83 %
HD 10	.323	.323	100 %	90 %
HD 12	.323	.323	100 %	95 %

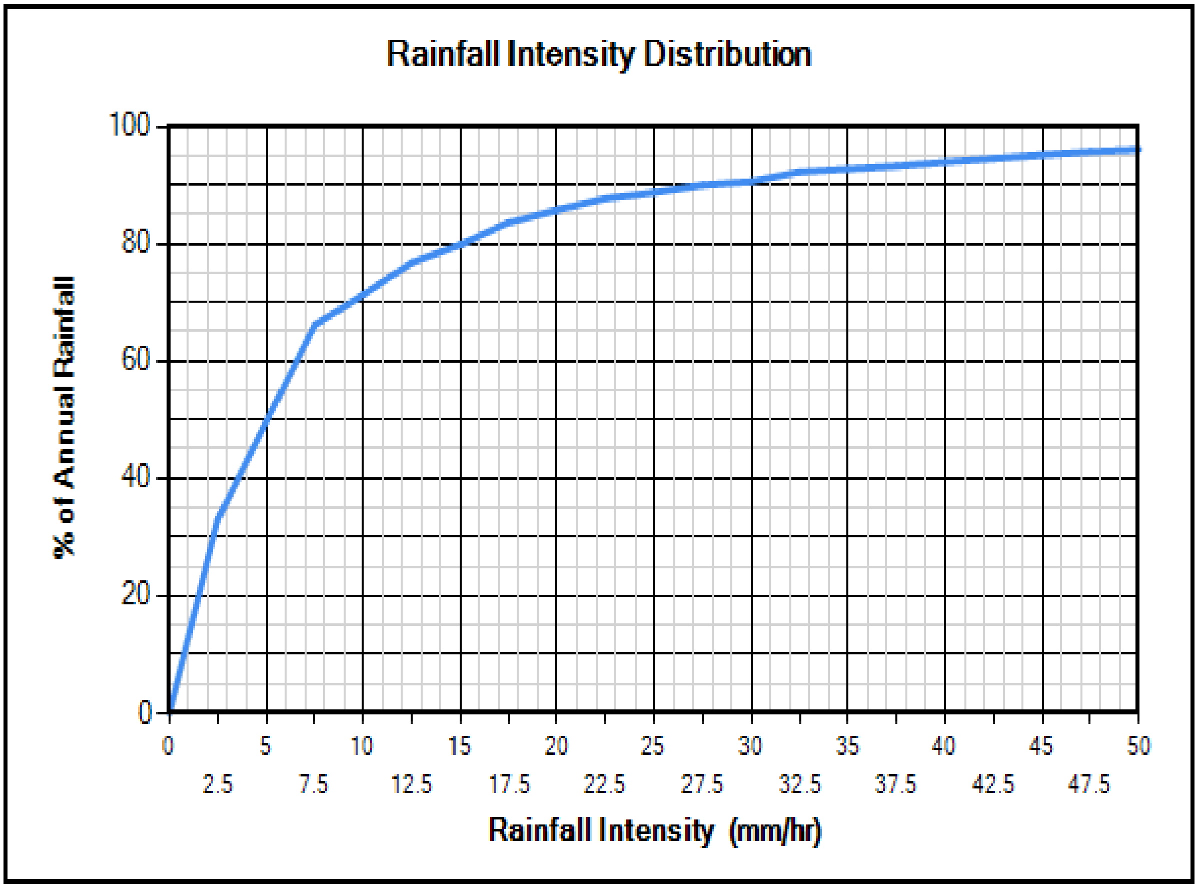
Size (um)	%	SG
20	35	2.65
35	10	2.65
63	5	2.65
88	10	2.65
125	15	2.65
200	15	2.65
325	5	2.65
750	5	2.65

TSS Particle Size Distribution

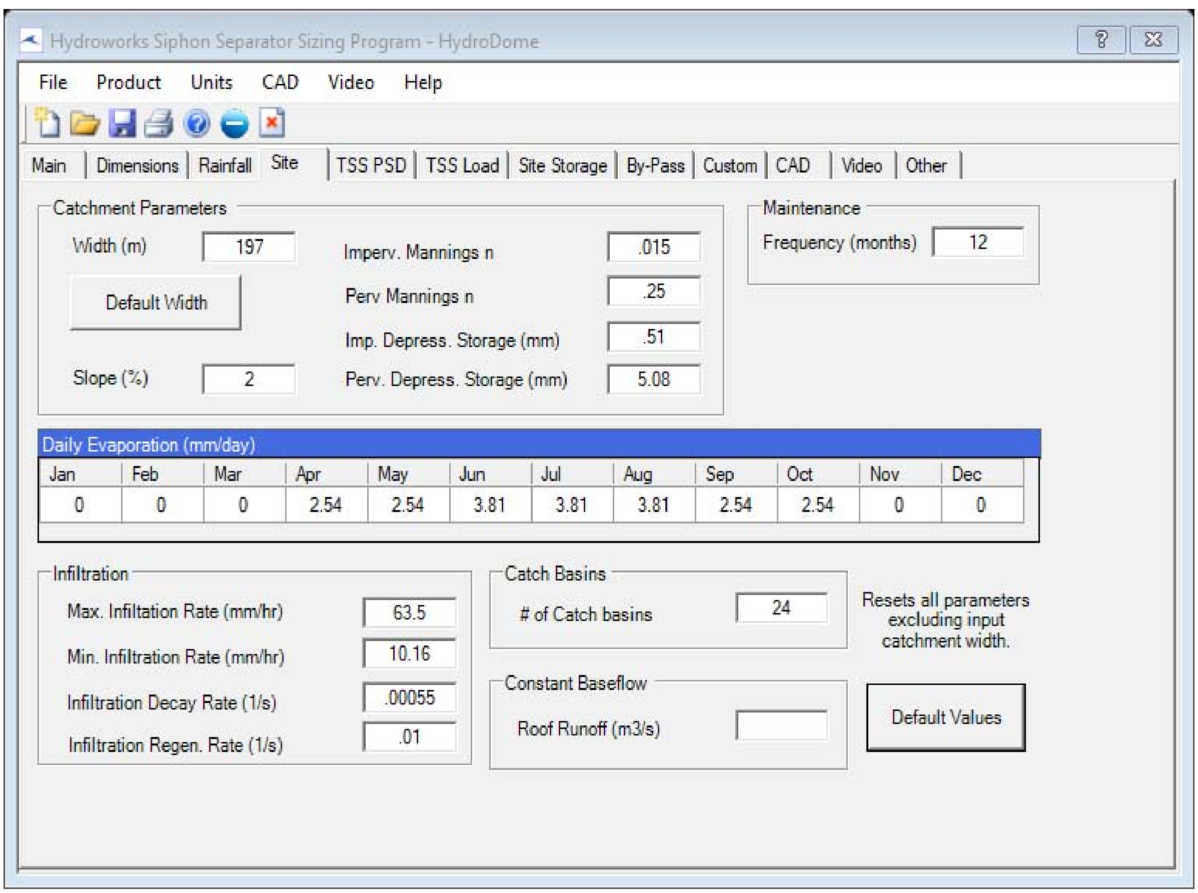


Size (um)	%	SG
20	35	2.65
35	10	2.65
63	5	2.65
88	10	2.65
125	15	2.65
200	15	2.65
325	5	2.65
750	5	2.65

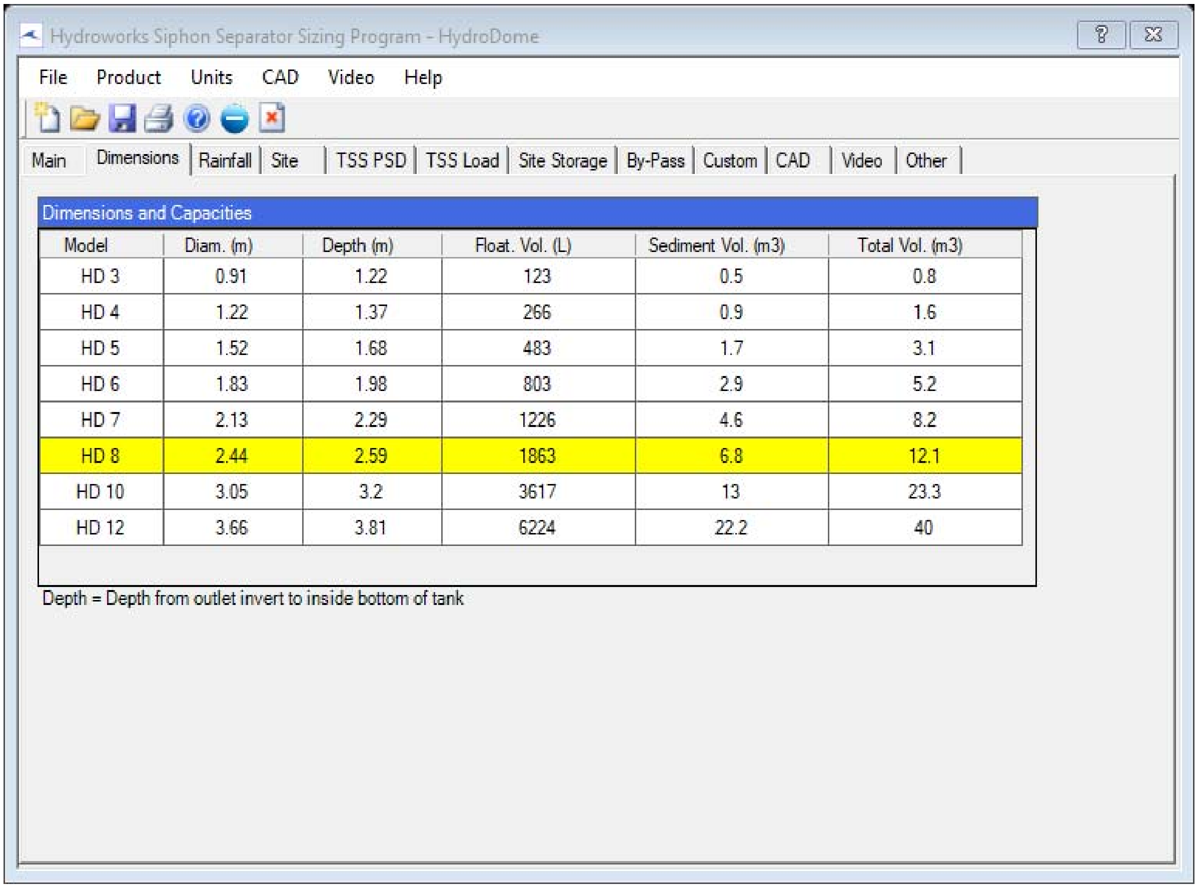
Rainfall Station - Toronto Bloor St., Ontario(1939 To 1986)



Site Physical Characteristics



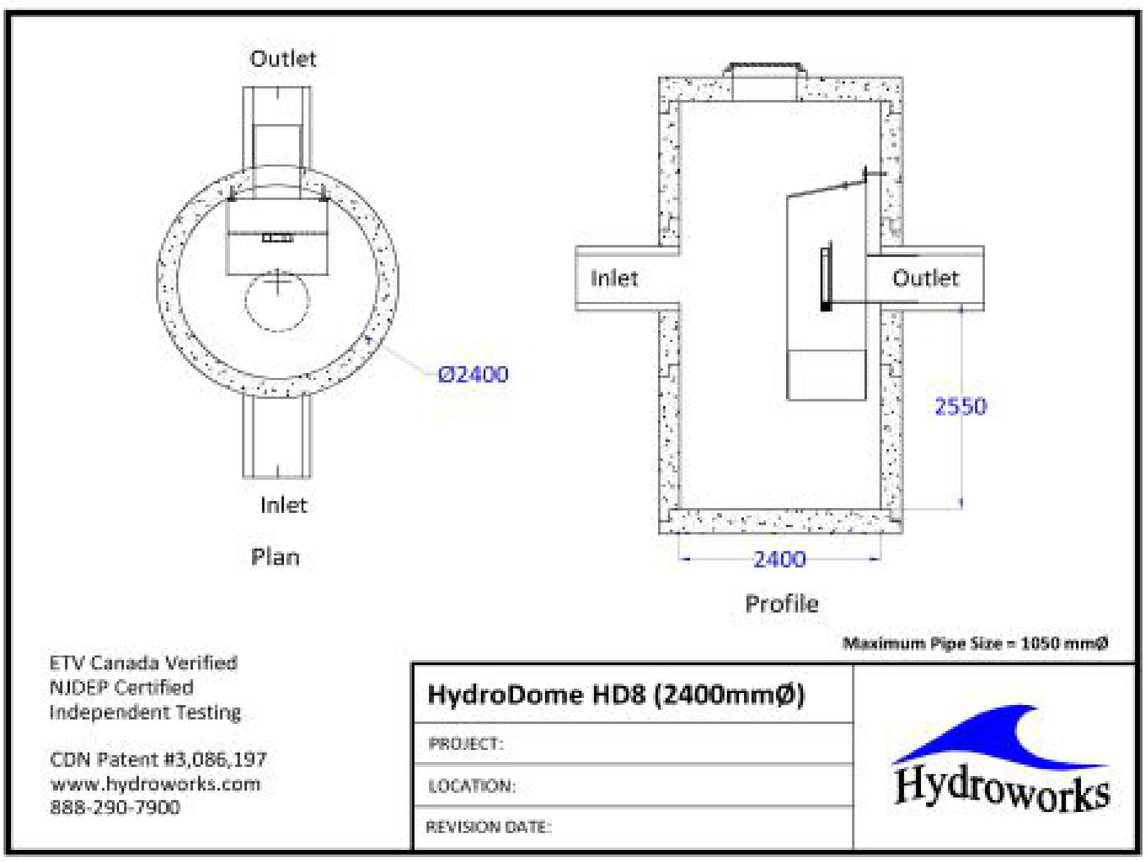
Dimensions And Capacities



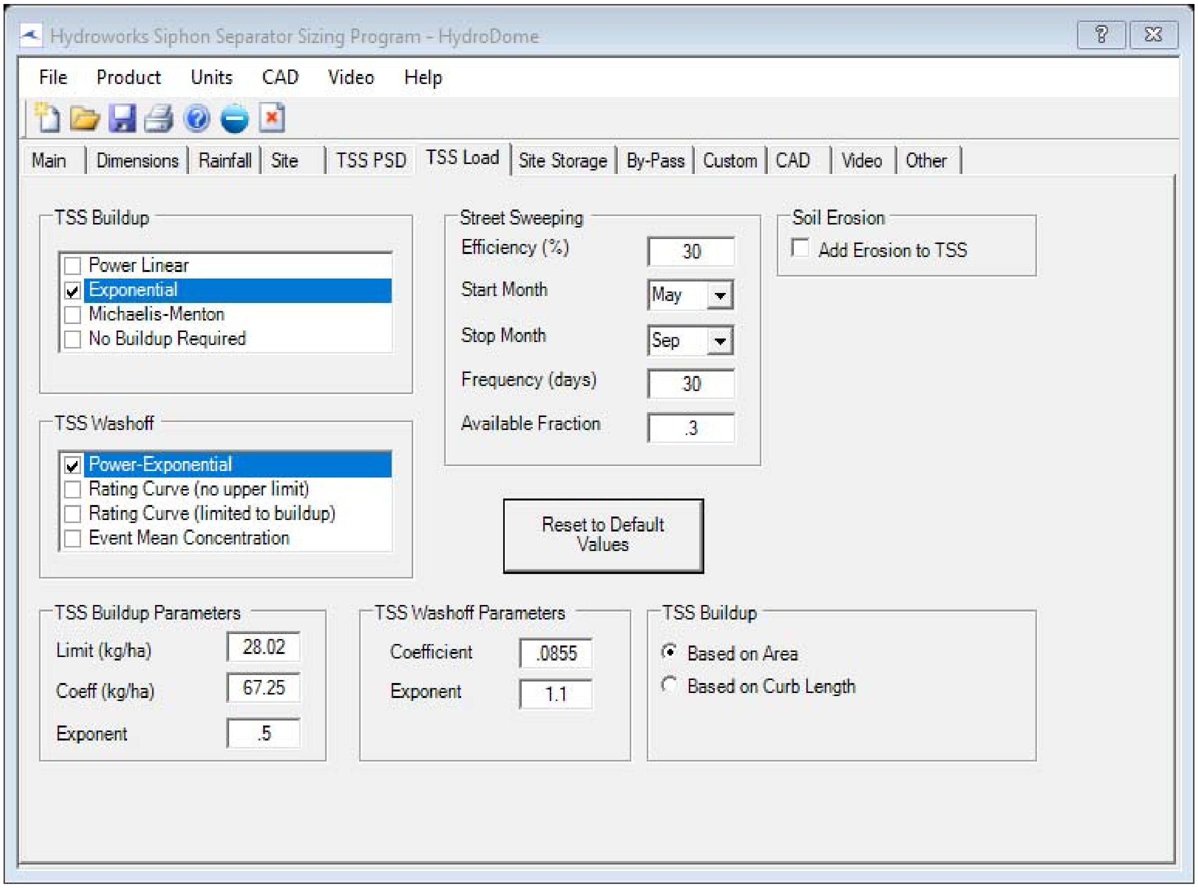
Model	Diam. (m)	Depth (m)	Float. Vol. (L)	Sediment Vol. (m3)	Total Vol. (m3)
HD 3	0.91	1.22	123	0.5	0.8
HD 4	1.22	1.37	266	0.9	1.6
HD 5	1.52	1.68	483	1.7	3.1
HD 6	1.83	1.98	803	2.9	5.2
HD 7	2.13	2.29	1226	4.6	8.2
HD 8	2.44	2.59	1863	6.8	12.1
HD 10	3.05	3.2	3617	13	23.3
HD 12	3.66	3.81	6224	22.2	40

Depth = Depth from outlet invert to inside bottom of tank

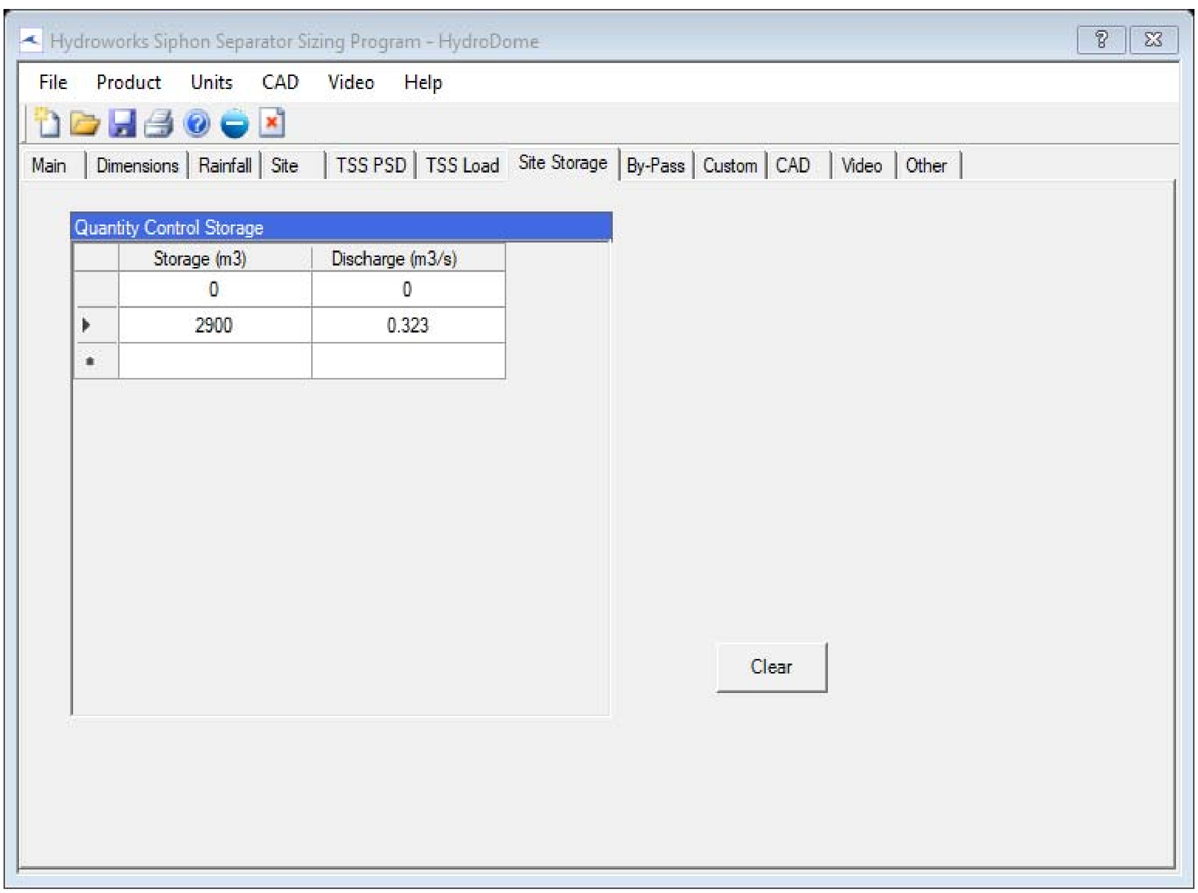
Generic HD 8 CAD Drawing



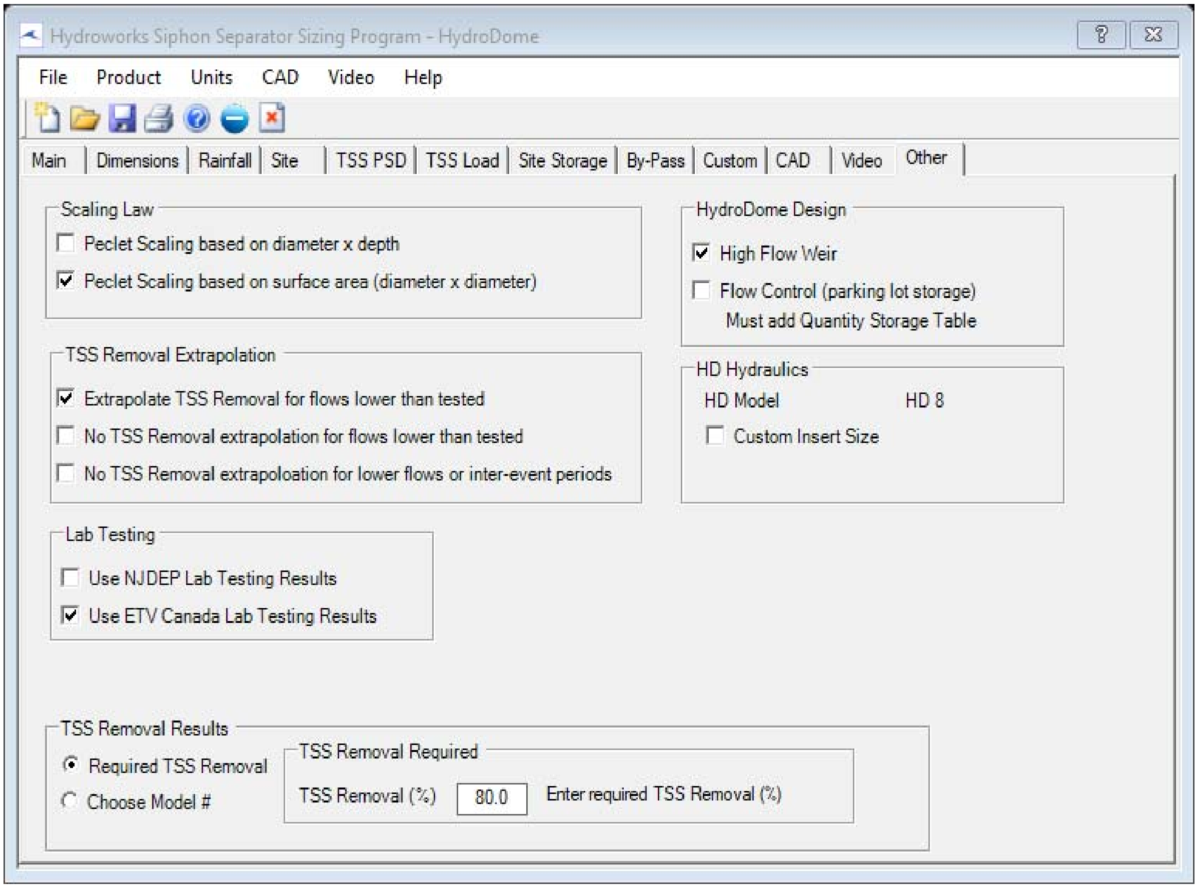
TSS Buildup And Washoff



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

APPENDIX C
Water Balance Calculations

Project Name:	DeZen Industrial Lands	**Prepared by:**	MJ
Municipality:	City of Mississauga	**Checked by:**	
Project No.:	220-M10	**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	**Prepared by:**	MJ
Municipality:	City of Mississauga	**Checked by:**	0
Project No.:	220-M10	**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**

Percent Increase Volume = (Total Post Vol. / Total Pre Vol.) x 100 — **87 %**

TOTAL VOL CHANGE FROM PRE- TO POST-: **5,581** **m^3/year**

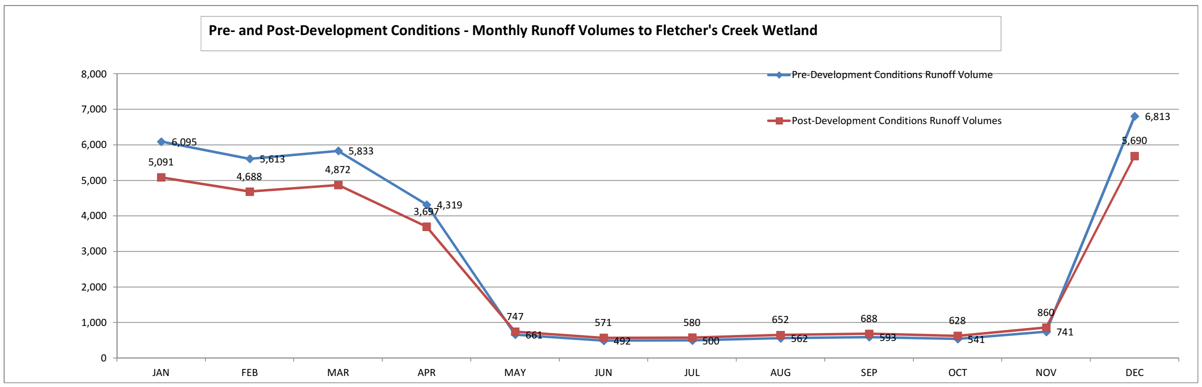
RUNOFF VOL CHANGE FROM PRE- TO POST-:	**4,499**	**m^3/year**	***Decrease***
RECHARGE VOL CHANGE FROM PRE- TO POST-:	**1,081**	**m^3/year**	***Decrease***

Pre-Development Conditions Monthly Runoff

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	YEAR
Runoff 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
Runoff 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 D

Sanitary Design Sheet
Sanitary Trunk Drawings

REGION OF PEEL

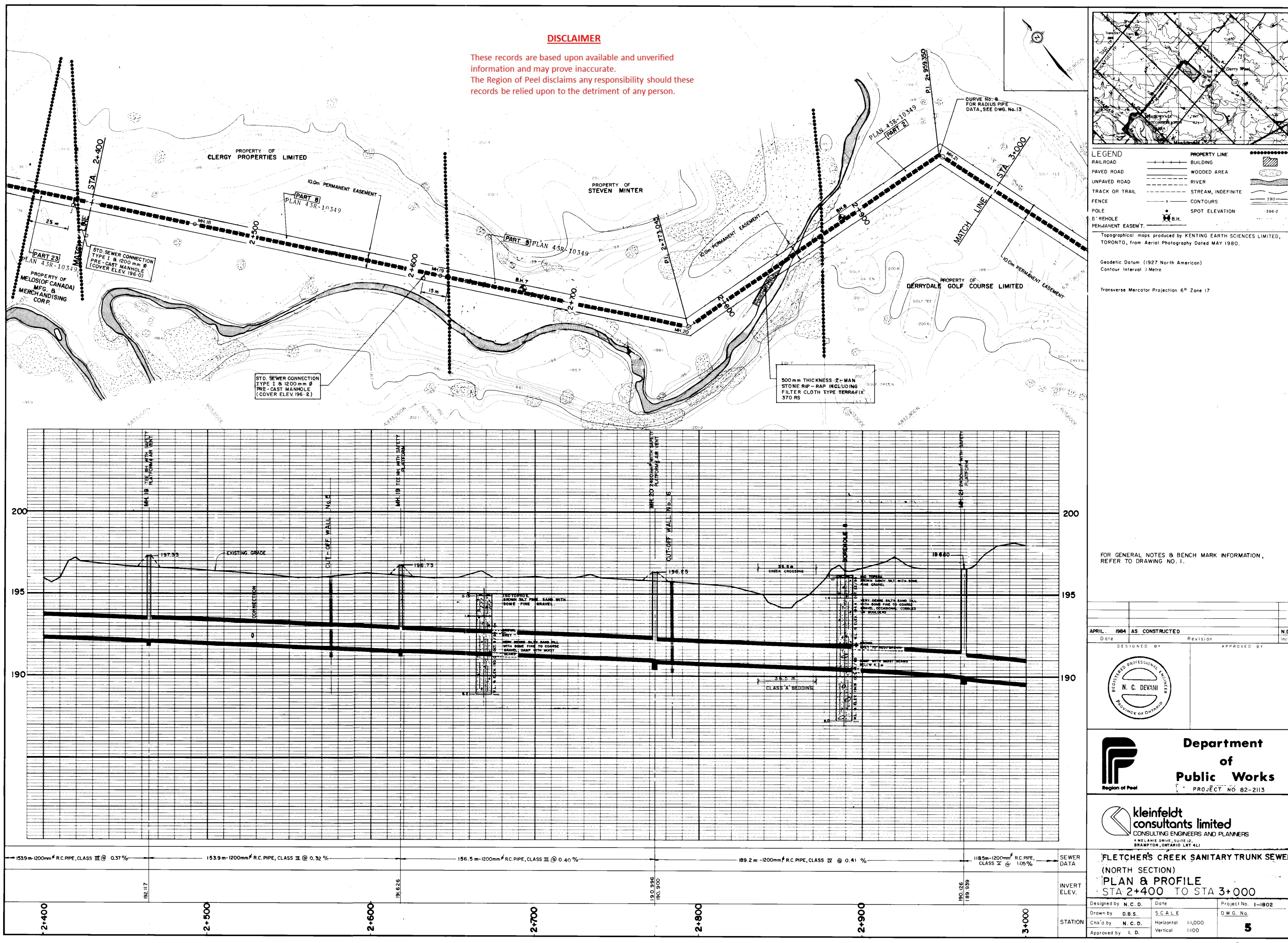
SANITARY SEWER DESIGN CHART

SUBDIVISION :	**DEZEN CONSTRUCTION COMPANY LIMITED**	SHEET No.	1 of 1
REGION FILE:		PROJECT No. :	220-M10
CONSULTANT :	**SKIRA & ASSOCIATES LTD.**	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.



9190-D

APPENDIX E
Stormwater Management Report
By Sernas

SERNAS ASSOCIATES
A Member of The Sernas Group Inc.

141 Brunel Road
Mississauga, ON
L4Z 1X3

T·416·213·7121
F·905·890·8499
sernas.com

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

Land Development Engineering
Land Development Planning
Municipal Engineering Services
Transportation & Transit Planning
Utility Infrastructure Design
Water Resources Engineering

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

Muneef Ahmad
for Ken Chow, P.Eng.
Principal, Manager – Water Resources

KC: ma

Encl.

c.c. Sernas Associates, Attn: Muneef Ahmad

TABLE OF CONTENTS

1.0 INTRODUCTION

The City of Mississauga has retained Sernas 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:

<u>Advantages</u>	<u>Disadvantages:</u>
Complete topographic information compiled	Cost may be prohibitive if not done on widescale basis
Faster than ground survey	
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

LICENSED PROFESSIONAL ENGINEER
M.M. AHMAD
04/04/03
PROVINCE OF ONTARIO

Muneef Ahmad, P.Eng.
Water Resources Engineer

FOR
Ken Chow, P.Eng.
Principal, Manager – Water Resources