URBANTECH®

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

3085 Hurontario

City of Mississauga

Prepared for

Equity Three Holdings Inc.
Project #: 20-653

June 2021

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1. INTRODUCTION

Urbantech has been retained as consulting engineers by Equity Three Holdings Inc. to complete a Functional Servicing Report in support of an official plan amendment and zoning bylaw amendment for the proposed 1.46 ha development located at 3085 Hurontario Street in the City of Mississauga.

As shown on **Figure FIG-1,** the site is bounded:

- To the north by commercial buildings and Kirwin Avenue
- To the south by commercial and residential buildings
- To the east by residential buildings and Jaguar Valley Drive
- To the west by Hurontario Street

The site is comprised of Lot 9 in registered plan TOR-12, Lot 15, Concession 1 North of Dundas Street and Block A Registered Plan 645 as shown on R-PE Surveying Sketch Showing Elevations, dated November 25th, 2020.

The site is currently occupied by commercial businesses and a parking garage.

The subject development lies within the limits of the Lake Ontario Shoreline East (Cooksville Creek) subwatershed, under the Credit Valley Conservation Authority (CVC) jurisdiction. The site falls within the City of Mississauga Hurontario/Main Street Corridor Master Plan area.

1.1 Study Purpose

The objective of this study is to outline the servicing requirements of the subject lands at a functional design level. This study will:

1. Recommend site grading, water supply and wastewater servicing strategies for the site.
2. Demonstrate compliance with City, Conservation and MECP design criteria for municipal services and stormwater management (SWM) measures.

The functional servicing design has been prepared in accordance with design criteria and requirements of the City of Mississauga, Region of Peel and Credit Valley Conservation Authority. The information in this report is intended to assist the regulatory agencies in their review of the planning applications for the proposed development.

2. DEVELOPMENT CONCEPT

Refer to the development concept plan **Drawing DP-1** prepared by Diamond Schmitt Architects Inc. The development plan consists of:

1. 31-storey building with 601 units.
2. 6-storey building with 136 units.
3. 25-storey building with 306 units and 0.15 ha of mixed-use space on the first floor.
4. 4 levels of underground parking.
5. 0.1 ha of private road area.
6. 0.3 ha of Privately Owned Publicly Accessible Spaces (P.O.P.S)
7. 0.15 ha of private garden area

The proposed development will connect to both Hurontario Street and Kirwin Avenue via private driveways.

2.1 Background Studies

The servicing and development concepts presented within this report are an extension of the information contained in the following reports:

1. Construction Dewatering Assessment – 3085 Hurontario Street (May 2020) by Hydrology Consulting Services.
2. Geotechnical Investigation – Proposed Condominium Development – 3085 Hurontario Street (May 2019) by Soil-Mat Engineers and Consultants Ltd.
3. Supplemental Geotechnical Investigation – Proposed Condominium Development – 3085 Hurontario Street (April 2020) by Soil-Mat Engineers and Consultants Ltd.
4. Cooksville Creek Flood Evaluation Master Plan EA (July 2012) by Aquafor Beech
5. Hurontario/Main Street Corridor Master Plan (October 2010) by MMM Group

3. EXISTING CONDITIONS

3.1 Land Use

The site is fully developed under existing conditions and consists of commercial businesses and an above grade parking garage.

3.2 Geotechnical and Hydrogeology

In support of the draft plan application, site specific soils and dewatering studies have been prepared by Soil-Mat Engineers and Consultants and Hydrogeology Consulting Services, respectively. The studies are reproduced in **Appendix E & F**.

The report by Soil-Mat Engineers and Consultants states that the site's soil stratigraphy is generally characterized by native clayey silt till overlain by native sand. Underlying bedrock was found to be Dundas Shale.

- The parking lot was found to consist of approximately 100 mm of asphaltic concrete overlying 200 mm of compact granular base.
- Beneath the pavement was loose brown sand with trace gravel up to a depth of 2.1 m.
- Stiff clayey silt overburden that was encountered up to a depth of 2.8-3.3 m below the existing grade.
- The overburden was underlain by highly weathered Dundas shale bedrock.

The Soil-Mat Engineers and Consultants report found the site's static groundwater level to be at a depth of approximately 3 to 3.6 m below the existing grade.

The report by Hydrology Consulting Services indicated the following:

- Discharging groundwater to municipal storm sewers would require treatment to reduce TSS concentrations, as well as TKN and E.coli/Fecal Coliform bacterial exceedances. This treatment will be developed by a dewatering contractor.
- Dewatering during construction would require a daily dewatering rate of 1,449,284 L/day during steady state conditions and a daily rate of 3,375,948 L/day when incorporating a factor of safety.
- A Permit to Take Water will be required for pumping during the excavation.

4. GRADING DESIGN

4.1 Design Standards

The proposed grading design for the site takes into consideration the following requirements and constraints:

1. Conforms to the City of Mississauga design criteria.
2. Match existing boundary lot and road grading conditions to be compatible with abutting properties.
3. Provides overland flow conveyance for major storm conditions.
4. Minimizes the need for retaining walls.
5. Provides appropriate cover on proposed servicing.
6. Ensures compatibility of driveway access to surrounding public streets.

4.2 Grading Design

A grading plan for the subject property has been prepared in conjunction with the storm, sanitary, and water servicing system design for the subject development.

Drawings GR-1 illustrate the proposed grading plan for the site.

Due to perimeter grading constraints adjacent to Hurontario Street and Kirwin, a minor portion of the site (0.051 ha) will drain uncontrolled to the public right of way.

5. STORM DRAINAGE AND STORMWATER MANAGEMENT

5.1 Drainage Criteria

The City of Mississauga and Credit Valley Conservation outline the following design criteria for the site as follows:

1. Meeting Cooksville Creek Subwatershed quantity control criteria of 100-year post development to 2-year predevelopment control.
2. Pre-development runoff coefficients are to not exceed 0.5 for a site that is already developed.
3. Ensure minimum 80% TSS removal on site for quality control.
4. First 5 mm of runoff to be retained on-site.
5. Provide safe overland flow conveyance of the 100-year event.

5.2 Storm Sewer Design

Storm sewers within the site will be sized to convey the 10-year storm in accordance with the City of Mississauga standards. The site is full coverage with underground parking. All surface drainage will be collected by area drains and catchbasins that are connected to the building plumbing system.

Storm sewer pipes will generally range in size from 200 mm to 525 mm diameter. Routing of the storm sewers within the building will be determined at a later date as the building design is advanced. All stormwater within the site is conveyed to the storage tank, which is situated in the south east corner of the site within the P1 level of the underground parkade.

There is an existing connection to the 525 mm storm sewer located at the south east corner of the site that will convey the controlled flows from the tank through an existing sewer to Jaguar Valley Drive.

Flows from 0.051 ha of the site along the boundary are not able to be captured by area drains and will flow uncontrolled towards Hurontario Street (0.046 ha) and Kirwin Avenue (0.005 ha).

5.3 Quality Control

As identified in section 5.1 above, the site is required to meet a minimum of 80% TSS removal on site for quality control.

To achieve the required TSS removal an Oil Grit Separator (OGS) will be used downstream of the proposed storage tank. The OGS device will be ETV certified to provide a minimum of 80% TSS removal. **Table 1** below outlines preliminary sizing for the OGS. Sizing specifications are to be verified by the manufacturer during detailed design.

Table 1: OGS Parameters

OGS #	Size	Area (ha)	Efficiency (%)
1	EFO8	1.402	81

Refer to **Appendix B** for the Stormceptor Sizing Report. Refer to Drawing STM-1 for the location of the OGS.

5.4 Quantity Control

The two-year pre-development flow from the site was established using the rational method and the Pearson International Airport IDF parameters. As the site is fully developed under existing conditions, a runoff coefficient of 0.5 was used as prescribed by the City of Mississauga standards. **Table 2** below outlines the pre-development 2-year flow.

Table 2 - 2-year Pre-development Target

Area (ha)	Runoff Coefficient	Rainfall Intensity (mm/hr)	2-year Target (m^3/s)
1.46	0.5	59.9	0.122

Storm water quantity control is achieved by using a storage tank with a 140 mm orifice located in the south east corner of the property. As noted in section 5.2 above, a 0.051 ha portion of the site drains uncontrolled to Hurontario Street, to account for this uncontrolled flow, the tank has been overcontrolled to ensure that the 2-year predevelopment target is not exceeded during the 100-year event. **Table 3** summarizes the flow and storage values required based on the rational method calculations.

Table 3: Flow and Required Storage Volume Results

Outlet	Area (ha)	Runoff Coefficient	Post Development Flows m^3/s	Required Volume (m^3)
Tank	1.411	0.9	0.099	551.5
Uncontrolled to Hurontario Street	0.051	0.9	0.022	-
		Total	0.122	551.5

As a tank of **624 m^3** is being provided there will be no increase from 2-year pre-development flows.

Refer to SWM Calculations in **Appendix B** for supporting calculations.

5.5 Water Balance/Water Re-use

As shown in section 5.1 above, the first 5 mm of a rain event are required to be retained onsite. For the site of 1.46 ha this results in a total volume of 73.1 m^3. Due to the high groundwater table and extent of the underground parking garage, there are no options for infiltrating the water. The storage tank will be designed to capture the first 5 mm for reuse. The end use of this water will be determined at detailed design.

Refer to the Water Balance in **Appendix B** for the supporting calculations.

6. WASTEWATER SERVICING

6.1 Existing Conditions

The existing sanitary sewer in proximity to the site is as follows:

1. 300 mm diameter located within Hurontario Street flowing south.

The location of the existing is sewer is shown on **Drawing SERV-**1.

The existing flows from the subject lands were calculated in accordance with Peel region standards and specifications, outlined in section 6.2 below. A conservative flow of 13.29 L/s was calculated.

Refer to Wastewater Demand Calculations in **Appendix C** for calculations.

6.2 Design Criteria

Wastewater sewers will be designed in accordance with Region of Peel standards and specifications. The following criteria were used:

- 3 people/unit for large apartments
- 50 people/ha for commercial areas
- 0.2 L/s/ha for infiltration
- 303 L/person/day for domestic sewage flow

6.3 Local Wastewater Design

The estimated sanitary flow from the subject lands is 37.9 L/s. Refer to Wastewater Demand Calculations in **Appendix C** for calculations.

Sanitary servicing within the site will be designed by the project mechanical engineer as the building design advances. Proposed sanitary flows from the site will be conveyed via a new 250 mm service connection to the existing public sewer on Hurontario Street. Refer to Drawing SERV-1 for the anticipated connection location.

The estimated flow was provided to the Region of Peel to verify sewer capacity using their model. The Region of Peel has advised that two lengths of public downstream sanitary sewer will be at or above capacity.

Peel Region indicated the following external upgrades may be required:

- replacement of 116m of existing 300mm located on Hurontario (north of Dundas); and,
- replacement of 15m of existing 375mm diameter sanitary sewer at Jaguar Valley at Dundas Street.

Consideration should be given to monitoring the capacity of the existing sewers to confirm the actual existing utilization. Due to the conservative nature of flow calculations and population estimates, the degree of surcharge could be overstated.

The Region indicated that these improvements would be at the developers' expense. Design and installation of the works on Hurontario would require coordination with the Region's LRT project.

7. WATER SERVICING

7.1 Existing Conditions

The existing water network, which falls under the jurisdiction of the Region of Peel, in the vicinity of the site includes:

1. A 400 mm local watermain on Hurontario Street
2. A 300 mm local watermain on Kirwin Avenue

7.2 Design Criteria

The proposed watermain design will comply with the Region of Peel design criteria as follows:

- Residential Consumption = 280 l/c/day, max day = 3
- Commercial Consumption = 300 l/employee/day, max day = 1.4
- Residential and Commercial Peak Hour = 3
- Minimum operating pressure = 40 psi
- Maximum operating pressure = 100 psi

7.3 Local Watermains

Water servicing will be provided to the site via two new water services as shown on Drawing S1. Peel Region has indicated that a redundant water service with isolation between the services will be required. The water service size is estimated to be 250 mm which will be confirmed as the project advances. Peel Region directed that the connection be made to the existing 300 mm watermain on Kirwin Avenue. The onsite water supply system will be designed by the project mechanical engineer as the building design advances.

The total proposed fire flow from the subject lands is estimated to be 133.3 L/s (2113 USGPM) and the domestic demand is 8.1 L/s (129 USGPM).

A hydrant flow test was conducted on the hydrant adjacent to the site on Hurontario Street as well as at 3094 Jaguar Valley Drive. The results of the test are shown in **Table 4**.

Table 4: Fire Flow Tests

Pressure (psi)	Flow (USGPM)
3085 Hurontario Street	
80.2	0
73.2	4725
20	15139
82.6	0
78.7	5586
20	25144

Water demand and the results of the hydrant flow test were provided to the Region of Peel and no water capacity constraints were found.

Refer to **Appendix D** for water demand calculation and hydrant flow test results.

8. EROSION AND SEDIMENT CONTROL AND CONSTRUCTION DEWATERING

Erosion and sediment controls measures as follows:

1. Installing heavy duty silt control fencing along the perimeter of the site at strategic locations.
2. Installing a temporary mud mat at the construction site entrance.
3. Wrapping the tops of all inlet structures with filter fabric and using install silt sacks.
4. Inspecting all sediment and erosion control controls to maintain them in good repair until such time as the Engineer or the City approves their removal.
5. Safe discharge of construction water in accordance with City and provincial guidelines.

Site-specific ESC and Groundwater disposal measures will be determined during the detailed design / site alteration application stage of the project.

9. CONCLUSIONS

This report has demonstrated that:

- The proposed site will be graded to match to existing elevations at all property lines. A retaining wall will be required at the south property limit.
- Building Storm drains will be designed by the project mechanical engineer at the building permit stage.
- Water quality will be provided through the use of an OGS device.
- Storm water quantity control estimated to be 551 m^3 will be required to control flows from the post development 100 year storm to the predevelopment 2 year storm in accordance with Mississauga standards.
- Storage will be provided with a tank located at the south-east corner of the building that will be integrated with the building parking structure.
- The site will utilize an existing 525mm storm sewer that serves the site and outlets to an existing storm sewer on Jaguar Valley Drive.
- Water balance objectives will be met by retaining the first 5 mm of rain events onsite in the storage tank. Retained water will be re-used for irrigation purposes.
- Wastewater servicing to the site will be provided by a new 250mm diameter connection to the 300mm diameter existing sewer on Hurontario Street.
- Peel Region has indicated that some of the existing sewers in the vicinity if the site may require capacity augmentation.
- Water servicing to the site will be provided by the existing 300 mm watermain on Kirwin as directed by the Region.
- Erosion and sediment control and groundwater control measures will be implemented during construction in accordance with City and Provincial requirements.

Report Prepared by:

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STEVEN A. HADER
7/8/2021
PROVINCE OF ONTARIO

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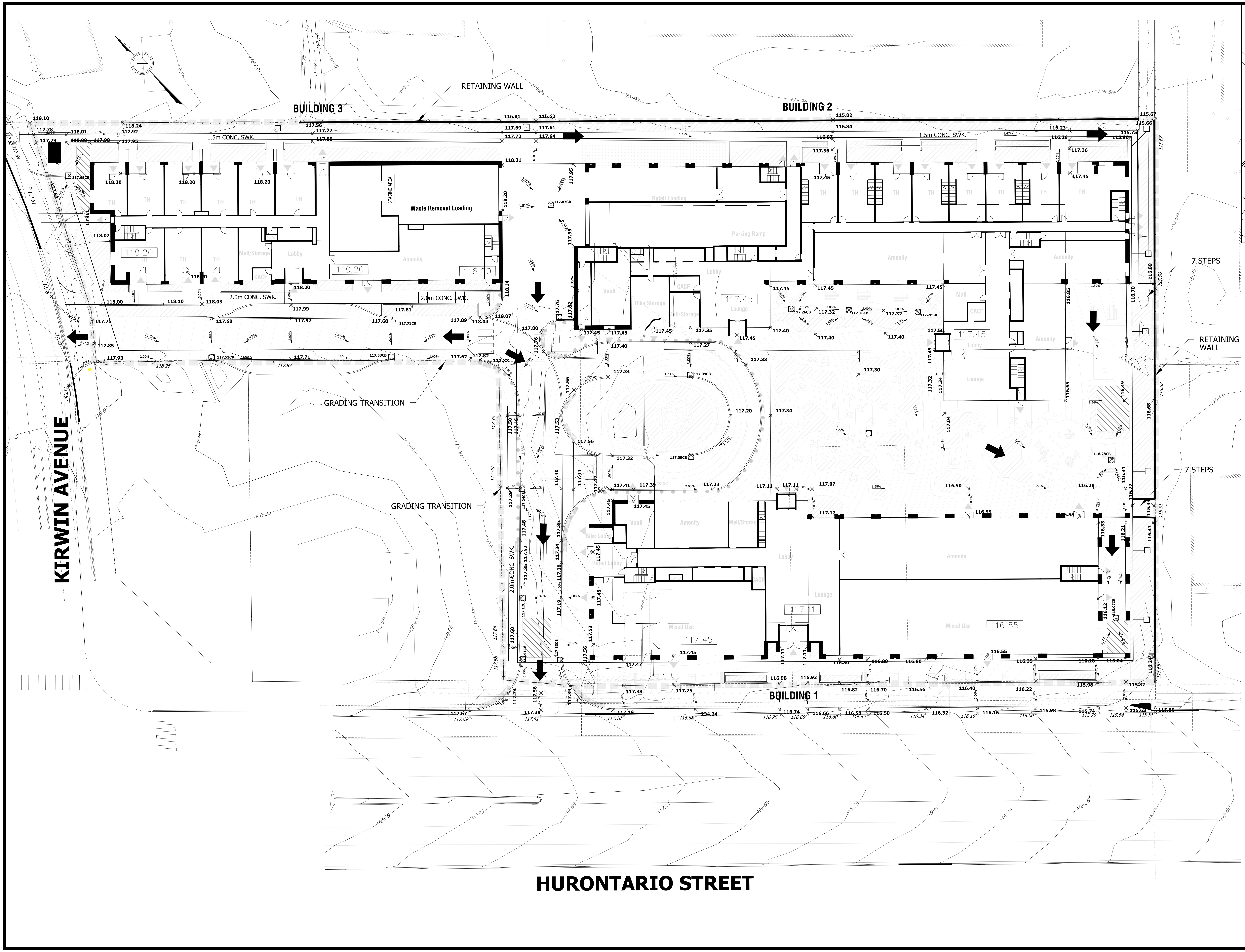
Municipal Design Assistant

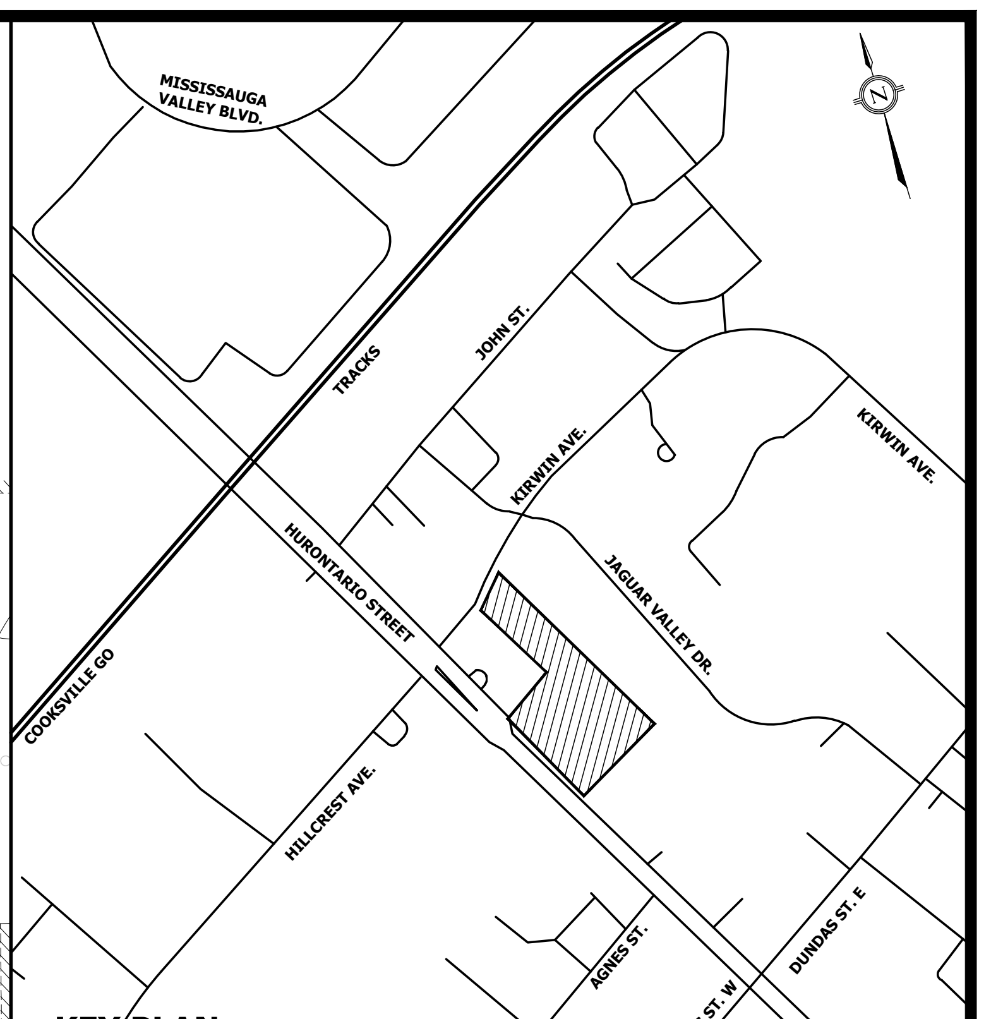
Steven Hader, P. Eng.
Senior Project Manager

APPENDIX A

Drawings and Figures

Figure FIG-1 Site Location Plan
Drawing GR-1 Grading Plan
Drawing STM-1 Storm Infrastructure
Drawing SERV-1 Sanitary and Water Infrastructure





KEY PLAN
N.T.S.

LEGEND

▭ ▭ ▭ ▭ ▭	LIMIT OF PROPERTY
92.50	EXISTING CONTOUR & ELEVATION
□	PROPOSED CATHBASIN
▣	PROPOSED AREA DRAIN
⌖ **90.50**	PROPOSED ELEVATION
⌖ **90.50CB**	PROPOSED AREA DRAIN/CB ELEVATION
⌖ *90.87*	EXISTING ELEVATION
┴┴┴┴	MAXIMUM 3:1 (UNLESS OTHERWISE NOTED)
➡	PROPOSED OVERLAND FLOW ROUTE

BENCHMARK

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO CITY OF MISSISSAUGA VERTICAL BENCH MARK NUMBER 1001 HAVING AN ORTHOMETRIC ELEVATION OF 123.221 METRES. ELEVATIONS ARE REFERENCED TO THE CANADIAN GEODETIC VERTICAL DATUM OF 1928, PRE-1978 ADJUSTMENT (CGVD: 1928: PRE-78ADJ.)

TABLET SET HORIZONTALLY AT THE BASE OF A 750MM CONCRETE TRAFFIC POLE ON THE SOUTH SIDE OF HILLCREST AVENUE, WEST OF CENTERLINE OF THE EASTERLY GO STATION ENTRANCE.

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3085 HURONTARIO STREET
CITY OF MISSISSAUGA

PRELIMINARY GRADING PLAN

PROJECT No.	DATE	SCALE	DWG No.
20-653	JULY 2021	1:300	**GR-1**

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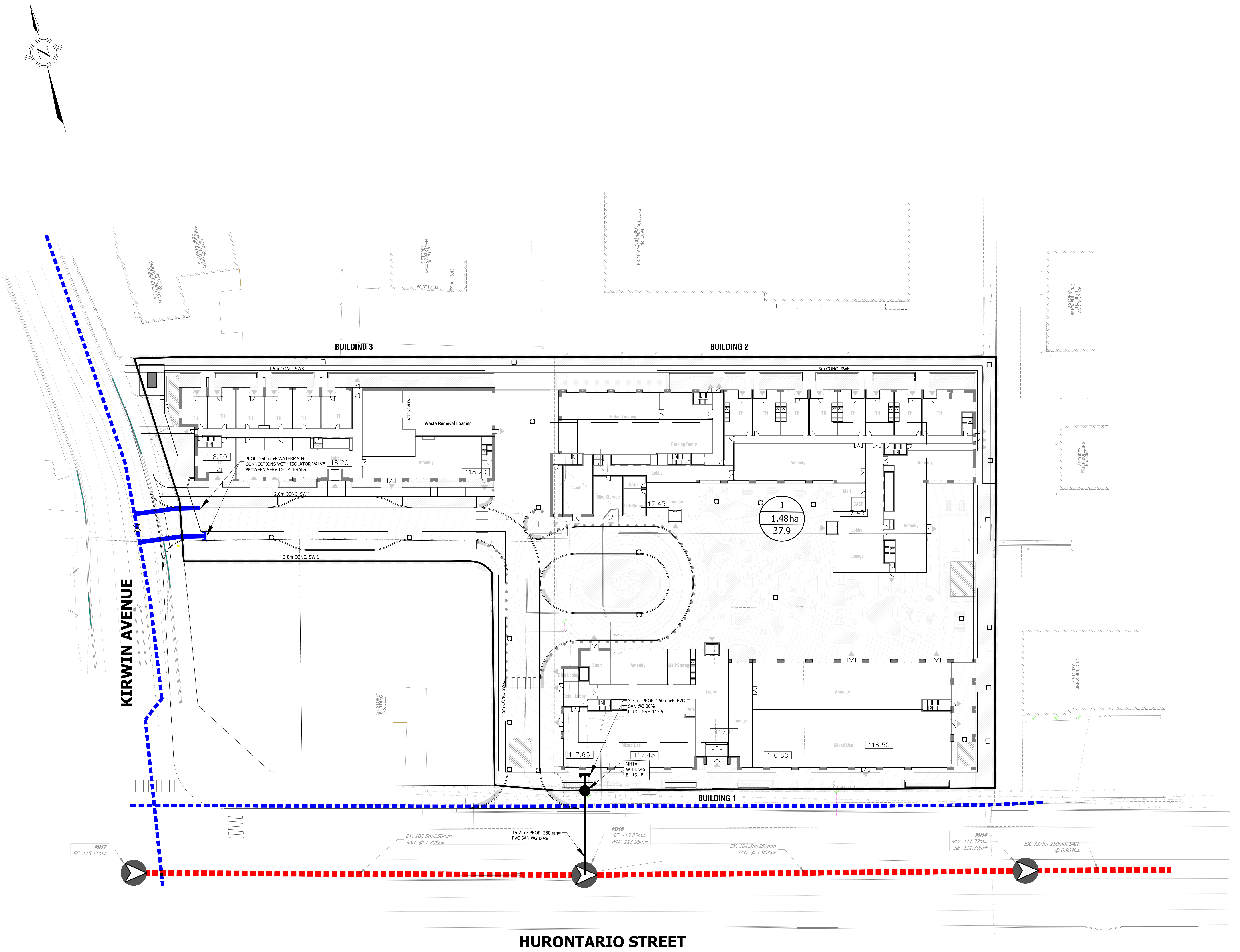

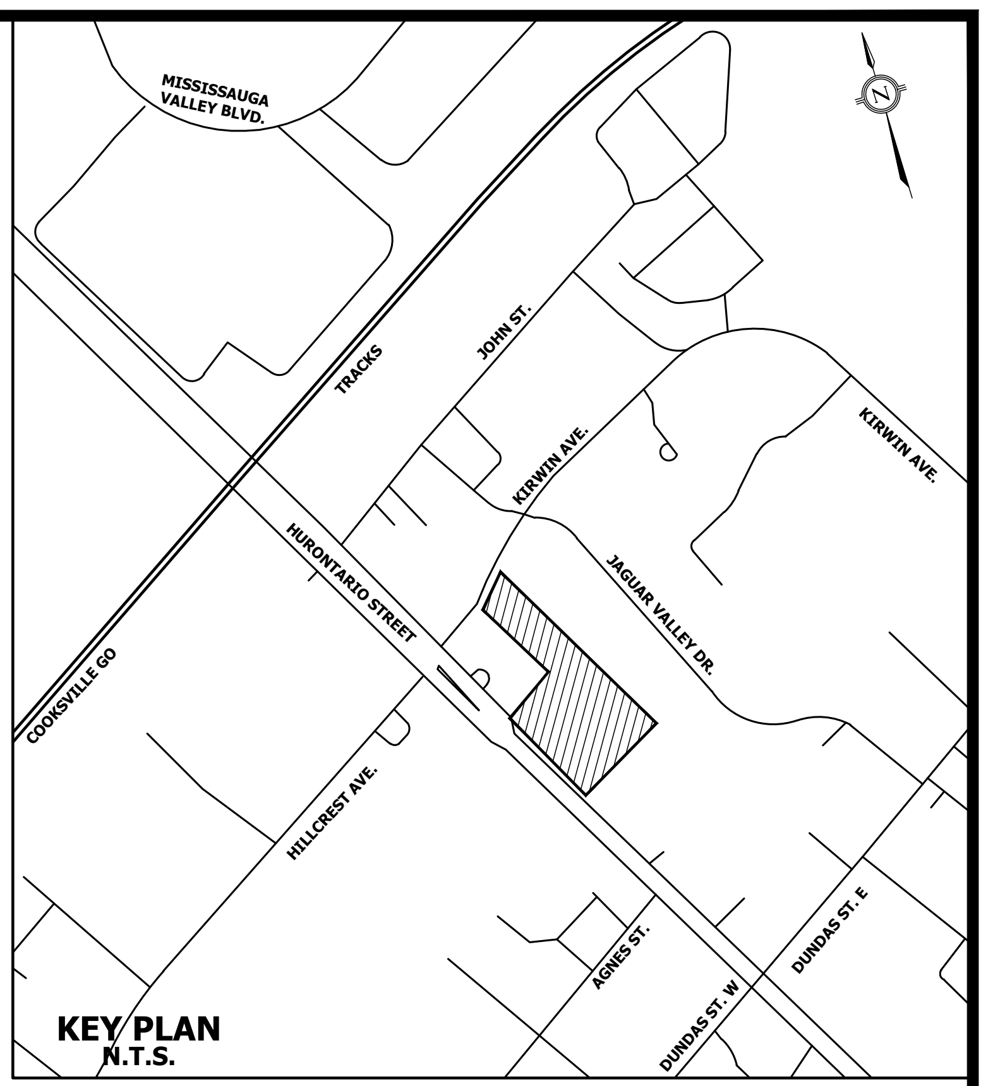

KEY PLAN
N.T.S.

LEGEND

- ▭ ▭ ▭ LIMIT OF PROPERTY
- ━━ SANITARY DRAINAGE AREA BOUNDARY
- ╌╌ EXTERNAL SANITARY DRAINAGE
- ━▶━ PROPOSED SANITARY SEWER
- ╌▶╌ EXISTING SANITARY SEWER
- AREA I.D. (49)
- AREA Ha. (0.68ha)
- TOTAL FLOW RATE (l/s) (60)
- SANITARY MANHOLE ID (SAN.MH5A)
- PROPOSED GROUND ELEVATION (TOP 90.75m)
- PROPOSED SEWER INVERTS (S 85.00m, N 85.00m, E 85.00m)
- ━━ PROPOSED PVC. WATERMAIN
- ╌╌ EXISTING WATERMAIN

BENCHMARK

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO CITY OF MISSISSAUGA VERTICAL BENCH MARK NUMBER 1001 HAVING AN ORTHOMETRIC ELEVATION OF 123.221 METRES. ELEVATIONS ARE REFERENCED TO THE CANADIAN GEODETIC VERTICAL DATUM OF 1928, PRE-1978 ADJUSTMENT (CGVD: 1928: PRE-78ADJ.)
TABLET SET HORIZONTALLY AT THE BASE OF A 750MM CONCRETE TRAFFIC POLE ON THE SOUTH SIDE OF HILLCREST AVENUE, WEST OF CENTERLINE OF THE EASTERLY GO STATION ENTRANCE.

116m OF EX. 300mm SANITARY SEWER ALONG HURONTARIO NORTH OF DUNDAS AND 15m OF EX. 375mm SANITARY SEWER ON JAGUAR VALLEY AT DUNDAS STREET WILL REQUIRE UPSIZING TO ACCOMMODATE THIS DEVELOPMENT. EXACT EXTENT OF REQUIRED WORKS SHALL BE COORDINATED WITH THE REGION.

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3085 HURONTARIO STREET
CITY OF MISSISSAUGA

PRELIMINARY SANITARY & WATERMAIN INFRASTRUCTURE

PROJECT No.	DATE	SCALE	DWG No.
20-653	JULY 2021	1:300	**SERV-1**

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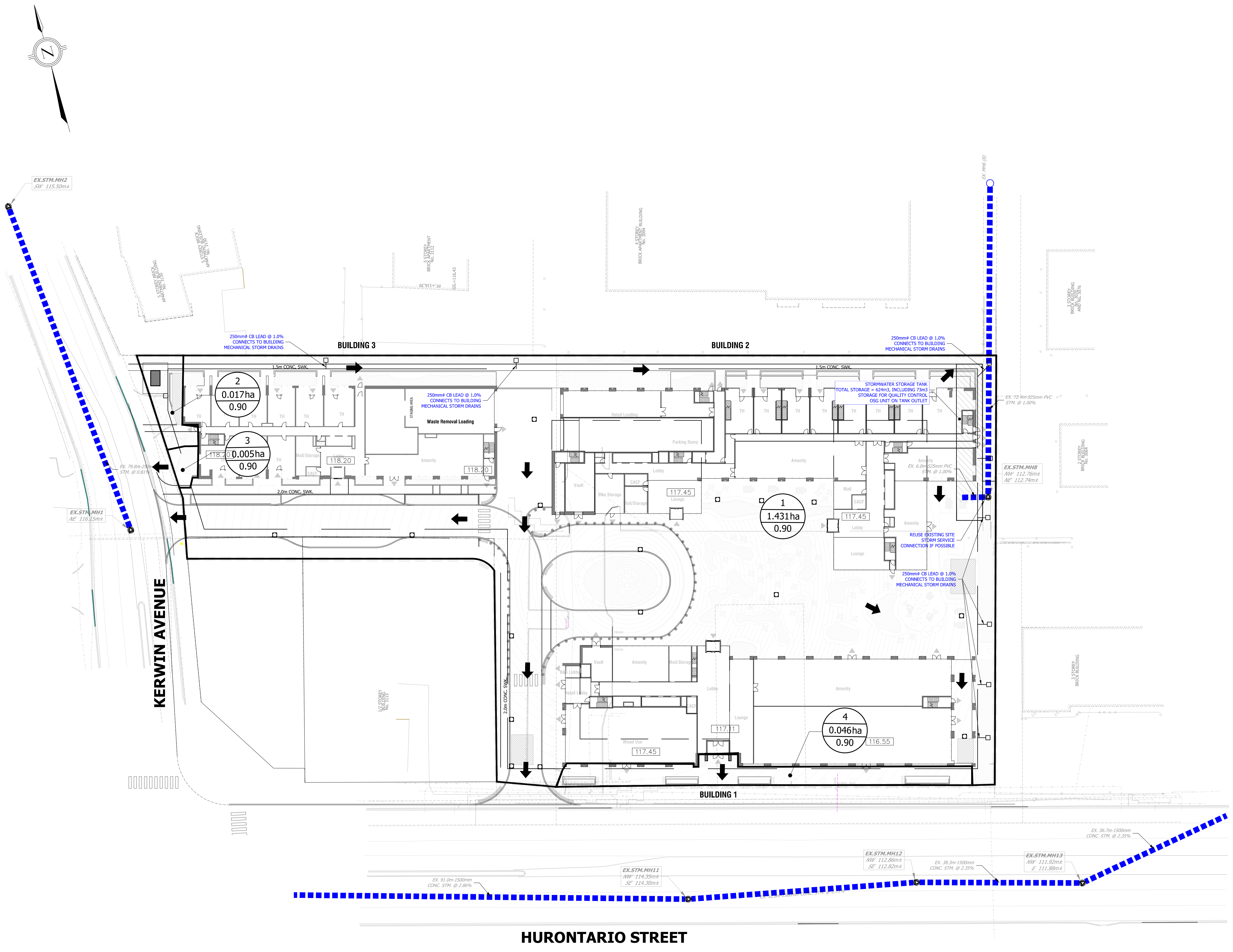

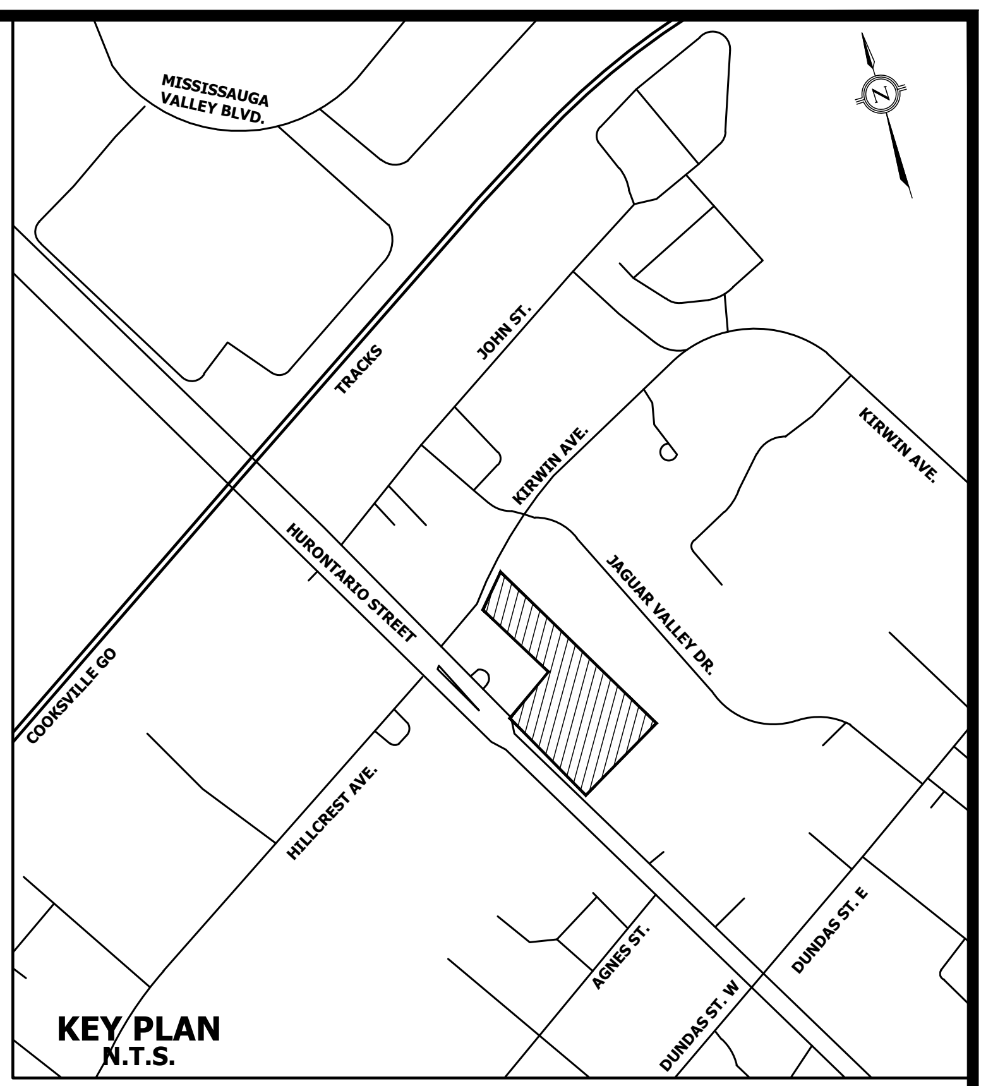

KEY PLAN
N.T.S.

LEGEND

▭ ▭ ▭	LIMIT OF PROPERTY
━━	STORM DRAINAGE AREA BOUNDARY
━▶━	PROPOSED STORM SEWER
╍▶╍	EXISTING STORM SEWER
41	AREA I.D.
0.26ha	AREA Ha.
0.65	RUNOFF COEFFICIENT
STM.MH2	STORM MANHOLE ID
TOP 90.70m	PROPOSED GROUND ELEVATION
S 85.00m E 85.00m W 85.00m N 85.00m	PROPOSED SEWER INVERTS
➡	PROPOSED OVERLAND FLOW ROUTE

BENCHMARK

ELEVATIONS ARE GEODETIC AND ARE REFERRED TO CITY OF MISSISSAUGA VERTICAL BENCH MARK NUMBER 1001 HAVING AN ORTHOMETRIC ELEVATION OF 123.221 METRES. ELEVATIONS ARE REFERENCED TO THE CANADIAN GEODETIC VERTICAL DATUM OF 1928, PRE-1978 ADJUSTMENT (CGVD: 1928: PRE-78ADJ.)
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3085 HURONTARIO STREET
CITY OF MISSISSAUGA

PRELIMINARY STORM INFRASTRUCTURE

PROJECT No.	DATE	SCALE	DWG No.
20-653	JULY 2021	1:500	**STM-1**

F:\PROJECTS\20-653 (3085 HURONTARIO -MISSISSAUGA-COOKSVILLE)\DRAWINGS\300 - DRAINAGE\20-653-STM.DWG

APPENDIX B

SWM Calculations

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SWM CALCULATIONS
ALLOWABLE OFFSITE RELEASE RATE

Project Name: 3085 Hurontario - Doracin
Municipality: City of Mississauga
Project No.: 20-653

Prepared by: JO
Checked by: SH
Date: 25-Jun-21

Site Area 1.46 ha

Pre-Development Runoff Coefficient

0.5 * 1.25 = 0.625

Proposed Discharge Location and Target Release Rate

Proposed Discharge Point	Target Rel. Rate (m^3/s)	
	2-Yr Pre	100-Yr Pre
To 525 mm in easement	0.122	0.357

Method of Determining Runoff: Rational Method, Q = 0.00278CIA

Where: Q = Peak flow rate (m^3/second)
C = Runoff coefficient
I = Rainfall intensity (mm/hour)
A = Catchment area (hectares)

Rainfall intensity per City of Mississauga Development Requirements (Section 8), $I = A/(T+B)^C$:

Where: A, B and C = Parameters defined in Mississauga Development Requirements Section 8.1
I = Rainfall intensity (mm/hour)
T = Time of concentration (hours)

Return Period (Years)	**2**	**100**
A	610	1,450
B	4.6	4.9
C	0.78	0.78
T (min) **	15	15
I (mm/hr)	59.9	140.7

** The minimum initial time of concentration is 15 minutes.

The 2-year pre-development flow rate is 0.122 L/s.
The 100-year pre-development flow rate is 0.357 L/s.

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SWM DESIGN CALCULATIONS
MODIFIED RATIONAL, 100-YR POST TO 2-YR PRE-DEV.

Project Name: 3085 Hurontario - Doracin
Municipality: City of Mississauga
Project No.: 20-653

Prepared by: JO
Checked by: SH
Last Revised: 6/25/2021

Target Release Rate - 2-Year Pre-Dev.		
525 STM	0.122	m^3/s
Uncontrolled	0.022	m^3/s
Max Tank Release	0.099	m^3/s

Post Development Condition:	Area (ha)	C
Controlled	1.411	0.90
Uncontrolled	0.051	0.90
	1.25*C=	1.13

IDF	A	B	C
100-Year	1,450	4.9	0.78

Time (min)	**Intensity 100-year** (mm/hr)	**Storm Runoff** (m^3/s)	**Target Release Rate** (m^3/s)	**Storage Accum. Rate** (m^3/s)	**Required Storage Volume** (m^3)
15	140.69	0.621	0.099	0.522	469.44
16	135.41	0.598	0.099	0.498	478.37
17	130.56	0.576	0.099	0.477	486.45
18	126.09	0.557	0.099	0.457	493.76
19	121.96	0.538	0.099	0.439	500.39
20	118.12	0.521	0.099	0.422	506.40
21	114.55	0.506	0.099	0.406	511.85
22	111.21	0.491	0.099	0.392	516.79
23	108.09	0.477	0.099	0.378	521.26
24	105.16	0.464	0.099	0.365	525.31
25	102.41	0.452	0.099	0.353	528.97
26	99.82	0.441	0.099	0.341	532.26
27	97.37	0.430	0.099	0.330	535.22
28	95.05	0.420	0.099	0.320	537.87
29	92.86	0.410	0.099	0.310	540.23
30	90.77	0.401	0.099	0.301	542.32
31	88.80	0.392	0.099	0.293	544.15
32	86.91	0.384	0.099	0.284	545.75
33	85.12	0.376	0.099	0.276	547.13
34	83.41	0.368	0.099	0.269	548.29
35	81.77	0.361	0.099	0.262	549.27
36	80.21	0.354	0.099	0.255	550.05
37	78.71	0.347	0.099	0.248	550.66
38	77.28	0.341	0.099	0.242	551.10
39	75.90	0.335	0.099	0.236	551.39
40	74.58	0.329	0.099	0.230	551.52
41	73.31	0.324	0.099	0.224	551.52
42	72.09	0.318	0.099	0.219	551.38
43	70.91	0.313	0.099	0.214	551.11
44	69.78	0.308	0.099	0.209	550.71
45	68.68	0.303	0.099	0.204	550.20
46	67.63	0.299	0.099	0.199	549.58
47	66.61	0.294	0.099	0.195	548.85
48	65.63	0.290	0.099	0.190	548.02
49	64.67	0.285	0.099	0.186	547.08
50	63.75	0.281	0.099	0.182	546.06
					551.52

SWM DESIGN CALCULATIONS
WATER BALANCE

Project Name: 3085 Hurontario - Doracin
Municipality: City of Mississauga
Project No.: 20-653

Prepared by: JO
Checked by: SH
Last Revised: 25-Jun-21

For this site, the minimum on-site runoff retention will require the site to retain all runoff from the first 5 mm of rainfall through infiltration, evapotranspiration or rainwater reuse, per CVC SWM Criteria (Section 4.2).

Site Area = 14618 m^2
Required Water Balance Volume = 73.1 m^3
Runoff Coefficient [1] = 0.9
Equivalent Imperviousness = 100% (based on $I = (C - 0.2) / 0.7$)

[1] Runoff Coefficient for Compact or dense housing (eg. Townhouses)
City of Mississauga, *Development Requirements Manual, Section 8*

Proposed Site Area Breakdown			
Cover	A (m^2)	IA (mm)	IA Volume (m^3)
Impervious	14,618	0	0.0
Pervious	0	0	0.0
Total	14,618		0.0

Total Initial Abstraction Volume = 0.0 m^3

Required Reuse Volume = SWM Tank Sump Volume
= 73.1 m^3

SWM DESIGN CALCULATIONS
ORIFICE DESIGN AND MINIMUM CISTERN SIZING

Project Name:	3085 Hurontario - Doracin	**Prepared by:**	JO
Municipality:	City of Mississauga	**Checked by:**	SH
Project No.:	20-653	**Last Revised:**	25-Jun-21

Orifice Control to Galesway Boulevard

Peak Discharge rate at maximum head, $Q = Cd\ A\ (2g\ H)^{0.5}$

Orifice Tube Diameter =	140	mm
C_d =	0.82	
A =	0.0153	m
g =	9.81	m^2/s
H =	2.90	m
Q =	0.095	m^3/s
Target release rate =	0.122	m^3/s
Uncontrolled Flow =	0.022	m^3/s
Max. Allowable Orifice Release Rate =	0.099	m^3/s

The peak discharge at maximum head is lower than the allowable municipal release rate (0.122 m3/s). The flow rate to the municipal storm sewer system is 0.095 m3/s.

Minimum Cistern Sizing

Sump Storage =	73.1	m^3
Total Active Storage Required =	551.5	m^3
Total Cistern Volume Required =	624.61	m^3

APPENDIX C

Wastewater Servicing

URBANTECH®

WASTEWATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653

Prepared by: J.P.O
Checked by:
Last Revised: 25-Jun-21

Existing Conditions

1-Storey Commercial

Population density = 50 p/ha
Area = 1.46 ha
Population = 74 persons

Domestic Sewage Flow = 13.00 L/s
*Per Region of Peel standards, for less than 1000 persons 0.013m^3/s should be used
Site Area = 1.46 ha
Infiltration Allowance = 0.20 L/s/ha
Total Infiltration = 0.29 L/s

Total wastewater flow = 13.29 L/s

URBANTECH®

WASTEWATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653

Prepared by: J.P.O
Checked by:
Last Revised: 25-Jun-21

Proposed Conditions

Residential

	# of Units	PPU
Building 1 =	601	3
Building 2 =	136	3
Building 3 =	306	3

Total Units = 1043
Population = 3129 persons

Harmon Peak Factor for Site, Me = $(1+14/(4+P^{0.5})$
3.43
Unit Sewage Flow = 303 L/person/day
Domestic Sewage Flow = 37.58 L/s

Mixed Use - Building 3
Population Denstity = 50 p/ha
Area = 0.15 ha
Population = 7.56 persons
Unit Sewage Flow = 303 L/person/day
Domestic Sewage Flow = 0.03 L/s

Site Area = 1.46 ha
Infiltration Allowance = 0.20 L/s/ha
Total Infiltration = 0.29 L/s

Total wastewater flow = 37.90 L/s

Increase in wastewater flow = 24.60 L/s

APPENDIX D
Water Servicing

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

Fire Flow Calculations

Based on the *Water Supply for Public Fire Protection, 1999* by Fire Underwriters Survey

1 Estimate of Fire Flow

F = 220 C (A)1/2

F = Fire Flow (L/min)
C = Construction Type Coefficient
= 0.6 ,for fire-resistive construction (fully protected frame, floors, roof)
A = Total flow area (m^2)
= If vertical openings and exterior vertical communications are properly protected (one hour rating),
Largest Floor + 25% of two immediately adjoining floors

Building 1

Floor	Area (m^2)	%
Level 1	3,515	100%
Level 1 Mezzanine	1,445	25%
Level 3	2,910	25%

= 4604 m^2

F = 8956 L/min
= 9000 L/min, rounded to the nearest 1000 L/min

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

---

2 Occupancy Reduction

15% for low hazard occupancies (apartments)
F = 7650 L/min

3 Sprinkler Reduction

30% for adequately designed sprinkler protection conforming to NFPA 13 and other NFPA sprinkler standards
F = 5355 L/min

4 Separation Charge

Direction	Separation (m)	Charge
North	17.4	15%
West	0.0	25%
South	0.0	25%
East	18.6	15%

Total Charge = 75%
F = 5738 L/min

Required Fire Flow

F = 11093 L/min
= 11000 L/min, rounded to the nearest 1000 L/min

Fire Flow Demand = 183.3 L/s
= 2906 USGPM

URBANTECH®

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

Domestic Flow Calculations

Population = 1623 persons, from Sanitary Calculations
Average Day Demand = 280 L/person/day, from Region of Peel design criteria
= 5.3 L/s

Use Peaking Factor the Greater of

Max Daily Demand PF = 2 , from Region of Peel design criteria
Max Daily Demand = 10.5 L/s

or

Max Peak Hour PF = 3 , from Region of Peel design criteria
Max Peak Hour Demand = 15.8 L/s

Domestic Flow Demand = 15.8 L/s
= 250 USGPM

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

Fire Flow Calculations

Based on the *Water Supply for Public Fire Protection, 1999* by Fire Underwriters Survey

1 Estimate of Fire Flow

F = 220 C (A)1/2

F = Fire Flow (L/min)
C = Construction Type Coefficient
= 0.6 ,for fire-resistive construction (fully protected frame, floors, roof)
A = Total flow area (m^2)
= If vertical openings and exterior vertical communications are properly protected (one hour rating),
Largest Floor + 25% of two immediately adjoining floors

Building 1

Floor	Area (m^2)	%
Level 2	1,650	25%
Level 3	1,650	100%
Level 4	1,650	25%

= 2475 m^2

F = 6567 L/min
= 7000 L/min, rounded to the nearest 1000 L/min

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

2 Occupancy Reduction

15% for low hazard occupancies (apartments)

F = 5950 L/min

3 Sprinkler Reduction

30% for adequately designed sprinkler protection conforming to NFPA 13 and other NFPA sprinkler standards

F = 4165 L/min

4 Separation Charge

Direction	Separation (m)	Charge
North	17.4	15%
West		
South	13.6	15%
East	0.0	25%

Total Charge = 55%

F = 3273 L/min

Required Fire Flow

F = 7438 L/min

= 7000 L/min, rounded to the nearest 1000 L/min

Fire Flow Demand = 116.7 L/s

= 1849 USGPM

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

Domestic Flow Calculations

Population = 367 persons, from Sanitary Calculations
Average Day Demand = 280 L/person/day, from Region of Peel design criteria
= 1.2 L/s

Use Peaking Factor the Greater of

Max Daily Demand PF = 2 , from Region of Peel design criteria
Max Daily Demand = 2.4 L/s

or

Max Peak Hour PF = 3 , from Region of Peel design criteria
Max Peak Hour Demand = 3.6 L/s

Domestic Flow Demand = 3.6 L/s
= 57 USGPM

URBANTECH®

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

Fire Flow Calculations

Based on the *Water Supply for Public Fire Protection, 1999* by Fire Underwriters Survey

1 Estimate of Fire Flow

F = 220 C (A)1/2

F = Fire Flow (L/min)
C = Construction Type Coefficient
= 0.6 ,for fire-resistive construction (fully protected frame, floors, roof)
A = Total flow area (m^2)
= If vertical openings and exterior vertical communications are properly protected (one hour rating),
Largest Floor + 25% of two immediately adjoining floors

Building 1

Floor	Area (m^2)	%
Level 2	1,650	25%
Level 3	1,650	100%
Level 4	1,650	25%

= 2475 m^2

F = 6567 L/min
= 7000 L/min, rounded to the nearest 1000 L/min

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

2 Occupancy Reduction

15% for low hazard occupancies (apartments)

F = 5950 L/min

3 Sprinkler Reduction

30% for adequately designed sprinkler protection conforming to NFPA 13 and other NFPA sprinkler standards

F = 4165 L/min

4 Separation Charge

Direction	Separation (m)	Charge
North	0.0	25%
West	12.8	15%
South		
East	7.5	20%

Total Charge = 60%

F = 3570 L/min

Required Fire Flow

F = 7735 L/min

= 8000 L/min, rounded to the nearest 1000 L/min

Fire Flow Demand = 133.3 L/s

= 2113 USGPM

WATER DEMAND CALCULATIONS

Project Name: 3085 Hurontario Street
Municipality: City of Mississauga
Project No.: 20-653
Date: 25/Jun/21

Domestic Flow Calculations

Residential Population = 826 persons, from Sanitary Calculations
Residential Average Day Demand = 280 L/person/day, from Region of Peel design criteria
= 2.7 L/s
Commercial Population = 8 persons, from Sanitary Calculations
Commercial Average Day Demand = 300 L/employee/day, from Region of Peel design criteria
= 0.03 L/s

Use Peaking Factor the Greater of

Residential Max Daily Demand PF = 2 , from Region of Peel design criteria
Commercial Max Daily Demand PF = 1.4 , from Region of Peel design criteria
Max Daily Demand = 5.4 L/s

or

Max Peak Hour PF = 3 , from Region of Peel design criteria
Max Peak Hour Demand = 8.1 L/s

Domestic Flow Demand = 8.1 L/s
= 129 USGPM

Fire Flow Testing Report

FLOWMETRIX
INDU-TECH
PROCESS
WESTCAN

Residual Hydrant #	**HY2020343**
NFPA Colour Code	**BLUE**

RESIDUAL HYDRANT INFO.

HYDRANT #	HY2020343	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	80.2	psi
RESIDUAL PRESSURE	73.2	psi
PRESSURE DROP	6.99	psi
% PRESSURE DROP	8.7	% psi

DATE 22-Apr-21
TIME 9:50 AM

ADDRESS
3085 Hurontario St
Mississauga, ON
L5A 2G9

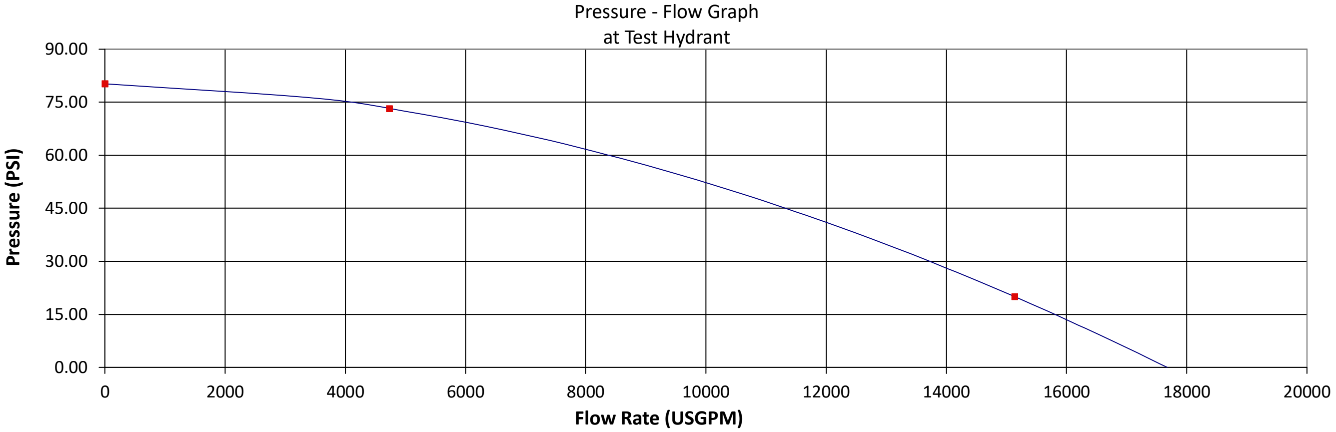
SIZE-inches/mm 16 400
CPP
Urbantech Consulting
Rob Merwin, P.Eng.
P : 905-829-6901
E : rmerwin@urbantech.com

Flow on Water Main At Test Hydrant - 20 psi 15139 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
HY2020342	2	2.5	Round	Swivel	1.00	49.8	2367	0
		2.5	Round	Swivel	1.00	49.8	2367	0
								0
								0
								0
								0
					Total Flow (USGPM)		4735	0
					Total Flow (USGPM)		4735	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Jordan Whitlock
OPERATOR	FMX	Denis Kriventsev
OPERATOR		Peel Region
PRESSURE ZONE		n/a
TOWER LEVEL	ft	n/a
PUMPS (ON/OFF)		n/a
OTHER-1		n/a
OTHER-2		n/a

Fire Flow Testing Report

Residual Hydrant # **HY6525644**
NFPA Colour Code **BLUE**

RESIDUAL HYDRANT INFO.

HYDRANT #	HY6525644	
N.F.P.A. COLOUR CODE	BLUE	
STATIC PRESSURE	82.6	psi
RESIDUAL PRESSURE	78.7	psi
PRESSURE DROP	3.86	psi
% PRESSURE DROP	4.7	% psi

DATE 22-Apr-21
TIME 10:10 AM

ADDRESS
3094 Jaguar Valley Drive
Mississauga, ON
L5A 2J4

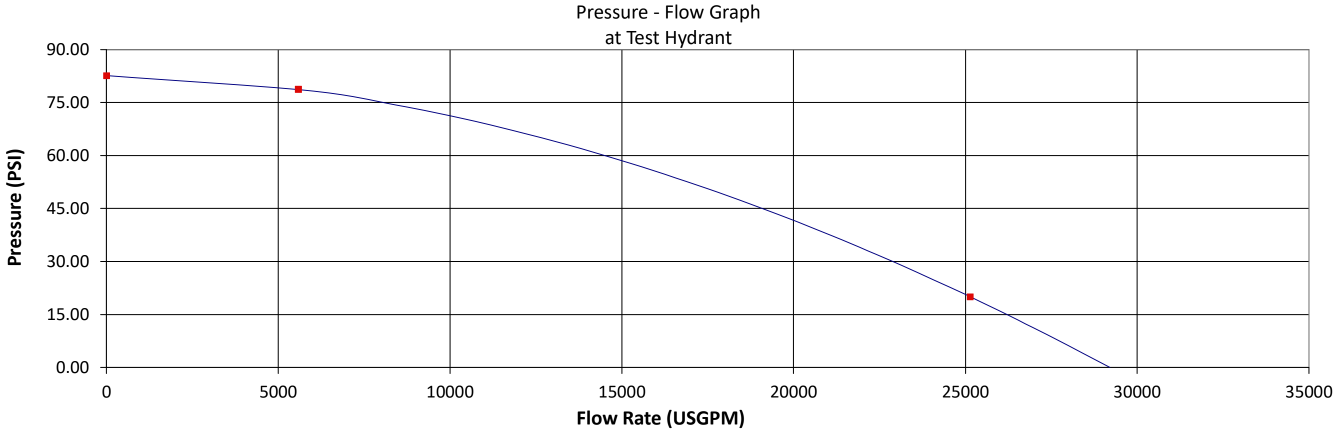
SIZE-inches/mm 12 300
PVC
Urbantech Consulting
Rob Merwin, P.Eng.
P : 905-829-6901
E : rmerwin@urbantech.com

Flow on Water Main At Test Hydrant - 20 psi 25144 USGPM

FLOW HYDRANT(S) INFO.

HYDRANT ASSET ID	HYD. # PORTS	OUTLET DIAMETER (INCHES)	NOZZLE COEFFICIENT	DIFFUSER TYPE	DIFFUSER COEFFICIENT	PITOT READING (psi)	PITOT FLOW (USGPM)	FLOW METER (USGPM)
HY6525624	2	2.5	Round	Swivel	1.00	69.3	2793	0
		2.5	Round	Swivel	1.00	69.3	2793	0
								0
								0
								0
								0
					Total Flow (USGPM)		5586	0
					Total Flow (USGPM)		5586	

FIRE FLOW CHART



COMMENTS

OPERATOR	FMX	Jordan Whitlock
OPERATOR	FMX	Denis Kriventsev
OPERATOR		Peel Region
PRESSURE ZONE		n/a
TOWER LEVEL	ft	n/a
PUMPS (ON/OFF)		n/a
OTHER-1		n/a
OTHER-2		n/a

APPENDIX E
Geotechnical Study

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

www.soil-mat.ca info@soil-mat.ca TF: 800.243.1922

Hamilton: 130 Lancing Drive L8W 3A1 T: 905.318.7440 F: 905.318.7455
Milton: PO Box 40012 Derry Heights PO L9T 7W4 T: 800.243.1922

PROJECT NO.: SM 190138-G May 8, 2019

OAKHILL ENVIRONMENTAL INC.
16 Walnut Street
St. Catharines, Ontario
L2T 1H3

Attention: Raivo Tahiste, B.Sc.

GEOTECHNICAL INVESTIGATION
PROPOSED CONDOMINIUM DEVELOPMENT
3085 HURONTARIO STREET
MISSISSAUGA, ONTARIO

Dear Mr. Tahiste,

Further to your authorisation, SOIL-MAT ENGINEERS & CONSULTANTS LTD. has completed the fieldwork, laboratory testing, and report preparation in connection with the above noted project. The scope of work was completed in general accordance with our proposal P7923 dated March 4, 2019. Our comments and recommendations based on our findings at the two [2] borehole locations are presented in the following paragraphs.

1. INTRODUCTION

We understand that the project will involve the construction of a condominium development consisting of four [4] condominium towers between 18 and 40 stories in height, with one shared common underground parking level, as well as above ground 4 to 6 storey 'podium' levels. The purpose of this geotechnical investigation work is to assess the subsurface soil, bedrock, and groundwater conditions, and to provide our comments and recommendations with respect to the design and construction of foundations for the proposed development, from a geotechnical point of view.

This report is based on the above summarised project description, and on the assumption that the design and construction will be performed in accordance with applicable codes and standards. Any significant deviations from the proposed project design may void the recommendations given in this report. If significant changes are made to the proposed design, this office must be consulted to review the new design with respect to the results of this investigation. It is noted that the information contained in this report does not reflect upon the environmental aspects of the site, which therefore have not been addressed in this document.

2. PROCEDURE

A total of two [2] sampled boreholes were advanced at the locations illustrated in the attached Drawing No. 1, Borehole Location Plan. It is noted that given the presence of existing buildings and parking structures, the relatively small on grade parking lot in the central portion of the site was the only area accessible with drill equipment at the time of our fieldwork. The boreholes were advanced using continuous flight power auger equipment under the direction of a representative of SOIL-MAT ENGINEERS to practical auger refusal on Dundas shale bedrock at depths of between 3.3 and 4.7 metres.

Representative samples of the subsoils were recovered from the borings at selected depth intervals using split barrel sampling equipment driven in accordance with the requirements of ASTM test specification D1586, Standard Penetration Resistance Testing. After undergoing a general field examination, the soil samples were preserved and transported to the SOIL-MAT laboratory for visual, tactile, and olfactory classifications. Routine moisture content tests were performed on all soil samples recovered from the borings, with hand penetrometer testing conducted on all cohesive samples.

Additionally, approximately 2.7 metres of the bedrock was cored in Borehole No. 1 using Nq diamond barrel coring equipment from approximately 5.2 to 7.9 metres. Recovered core samples of the bedrock were preserved and returned to the SOIL-MAT laboratory for testing including Rock Quality Designation [RQD] and unconfined compressive strength testing. The results of this testing can be found appended to the end of this report.

Upon completion of drilling, a groundwater monitoring well was installed at one borehole location to allow for future measurements of the static groundwater level. The monitoring well consisted of 50-millimetre diameter PVC pipe, screened in the lower 1.5 metres. The well was encased in well filter sand up to approximately 0.3 metres above the screened portion, then with a bentonite ‘hole plug’ up to the surface, and fitted with a protective steel monument casing. The remaining borehole was backfilled in general accordance with Ontario Regulation 903, and the ground surface reinstated with the existing grade using a pre-mixed ‘cold patch’ asphalt product.

The boreholes were located on site by a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD., based on accessibility over the site and clearance of underground services. The ground surface elevation at the borehole locations has been referenced to a site-specific temporary benchmark, described as the top of the water valve cover located adjacent to the parking lot, as illustrated in the Borehole Location Plan. This benchmark has been assigned an elevation of 100.00 metres for convenience.

Details of the conditions encountered in the boreholes, together with the results of the field and laboratory tests, are presented in Log of Borehole Nos. 1 and 2, following the text of this report. It is noted that the boundaries of soil types indicated on the borehole logs are inferred from non-continuous soil sampling and observations made during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and therefore should not be construed at the exact depths of geological change.

3. SITE DESCRIPTION AND SUBSURFACE CONDITIONS

The subject site is located at 3085 Hurontario Street in Mississauga, Ontario. The property is currently occupied by a one to two storey commercial plaza, two storey parking structure at the north end of the property, and on grade asphalt paved parking areas on the western portion. The site is bound by existing commercial properties and Kirwin Avenue to the to the north, a commercial plaza to the south, by existing apartment buildings to the east, and Hurontario Street to the west. This assumes a north-south orientation of Hurontario Street.

The subsurface conditions encountered at the borehole locations are summarised as follows:

Pavement Structure

Both boreholes were advanced through the pavement structure of the existing on grade parking lot, which was found to consist of approximately 100 to 150 millimetres of asphaltic concrete overlying 150 to 200 millimetres of compact granular base.

Sand

Native sand was encountered beneath the pavement structure at both borehole locations. The sand was brown in colour, contained trace gravel and occasional organics in the upper levels, and was generally in a loose to compact condition. Native sand was proven to depths of approximately 1.5 to 2.1 metres at both borehole locations.

Clayey Silt

Native clayey silt was encountered beneath the sand at all borehole locations. The native cohesive soils were grey in colour, contained trace to some sand, trace gravel, and were generally very stiff to hard in consistency. The native clayey silt was proven to practical auger refusal on bedrock at depths of approximately 3.2 and 2.5 metres below the existing ground surface in Borehole Nos. 1 and 2, respectively.

Dundas Shale Bedrock

Dundas shale bedrock was encountered beneath the cohesive clayey silt at both borehole locations, at depths of approximately 3.2 and 2.5 metres in Borehole Nos. 1 and 2, respectively. A review of available published information, as well as our experience in the area, indicates the bedrock to Dundas shale (Georgian Bay Formation). The Dundas shale bedrock is grey in colour, weathered in the upper levels, becoming more sound with depth, with occasional more resistant limestone/siltstone layers. It is noted that the upper levels of the Dundas shale bedrock are severely weathered, exhibiting characteristics of a very stiff to hard cohesive soil, such that the upper about 2 to 3 metres was able to be penetrated by auger equipment. As such, the transition from the very stiff to hard cohesive overburden clayey silt to weathered Dundas shale bedrock is somewhat indistinct. The depths to bedrock as encountered at the borehole locations has been summarised as follows:

TABLE A – BEDROCK DEPTH/ELEVATION

	Surface Elevation	Weathered Bedrock Depth	Bedrock Elevation
Borehole No. 1	100.03 m	3.2 m	97.8 m
Borehole No. 2	99.79 m	2.5 m	97.3 m

Note – these are relative elevations to a temporary benchmark, and are not geodetic.

As noted above, approximately 2.7 metres of the bedrock was cored in Borehole No. 1 from approximately 5.2 to 7.9 metres below the existing grade, using Nq diamond barrel coring equipment. The cores were noted to yield recoveries of approximately 100 per cent, with a Rock Quality Designation [RQD] of approximately 29% to 36%. Unconfined compressive strength testing on selected portions of the recovered samples yielded compressive strengths of approximately 15.3 to 18.9 MPa, with an average of 17.7 MPa. This is consistent with a highly weathered, weak rock or poor to moderate quality. The results of this testing can be found appended to the end of this report, and have been summarised as follows:

TABLE B – SUMMARY OF BEDROCK CORING

Borehole No. 1					
Depth of Core	Elevation of Core	Recovery	Rock Quality Designation (RQD)	Depth, Elevation of Tested Core Sample	Unconfined Compressive Strength
5.2 to 6.4 m	94.8 to 93.6 m	100%	29%	NA	NA
6.4 to 7.9 m	93.6 to 92.1 m	100%	36%	7.1 m, 92.9 m 7.8 m, 92.2 m	15.3 MPa 18.9 MPa

Groundwater Observations

Both of the boreholes were recorded as ‘dry’ upon completion of drilling. It is noted that insufficient time would have passed for the static groundwater level to stabilise in the open boreholes. As noted above, a monitoring well was installed at Borehole No. 2 to allow for future measurements of the static groundwater level. The water levels measured at this location have been summarised as follows:

TABLE C – SUMMARY OF GROUNDWATER MEASUREMENTS

	Surface Elevation	Groundwater Depth	Groundwater Elevation
April 24, 2019	99.79 m	3.1 m	96.7 m
May 7, 2019	99.79	3.0 m	96.8 m

Based on our observations during drilling and measurements of the groundwater level taken from the monitoring well, the static groundwater level is estimated at a depth of approximately 3 metres below the existing grade, at an elevation of approximately 96.8 metres, and would be expected to fluctuate seasonally.

It is noted that the above bedrock and groundwater elevations are based on a reference to a temporary benchmark with an assumed elevation of 100.00 metres. These elevations should be corrected upon determination of the geodetic elevation of the benchmark utilised.

4. FOUNDATION CONSIDERATIONS

With one underground parking level, it is anticipated that the proposed founding depth will be on the order of 3 to 4 metres, just into the weathered Dundas shale bedrock. Spread footings founded on the upper weathered levels of the Dundas shale bedrock may be designed using a factored Ultimate Limit State [ULS] bearing capacity of 1,500 kPa [~30,000 psf]. The bearing capacity at Serviceability Limit State [SLS] should be limited to 1,000 kPa [~20,000 psf]. Where foundations are founded on the competent Dundas shale bedrock, approximately 2 metres through the upper weathered levels, spread footings may be designed considering a bearing capacity of 1,500 kPa [30,000 psf] SLS and ULS. Higher bearing values would likely be available at greater depths into the more sound Dundas shale bedrock, however this would need to be confirmed by further assessment of the bedrock condition to greater depth. In any case, it is not expected that the available increase in bearing value would be sufficient to make the increased depth of foundation construction economical. The exposed footing beds must be hand cleaned of any loose or disturbed material, along with any ponded water, immediately prior to the placement of concrete.

It is noted that the SLS value represents the Serviceability Limit State, which is governed by the tolerable deflection [settlement] based on the proposed building type, using unfactored load combinations. The ULS value represents the Ultimate Limit State and is intended to reflect an upper limit of the available bearing capacity of the founding soils in terms of geotechnical design, using factored load combinations. There is no direct relationship between ULS and SLS; rather they are a function of the soil type and the tolerable deflections for serviceability, respectively. Evidently, the bearing capacity values would be lower for very settlement sensitive structure and larger for more flexible buildings. It is noted that the SLS and ULS bearing capacities are equivalent within the competent Dundas shale bedrock, as in order for the serviceability limit states to be realised, ultimate failure of the bedrock would have to occur.

It may also be considered to support the high-rise buildings on raft slab foundations on the weathered shale bedrock. Raft slabs may be designed using a rigid method considering the bearing values recommended above. If a flexible design method is used, a modulus of subgrade reaction of $k = 80 \text{ MN/m}^3$ [~290 pci] may be considered.

The support conditions afforded by the founding soils are usually not uniform across the site, nor are the loads on the various foundation elements. It is therefore recommended that the footings and foundation walls be structurally reinforced to account for potential variable support and loading conditions.

In areas where it will be necessary to provide adjacent footings at different founding elevations, the lower footing should be constructed before the higher footing is constructed, if possible, and the higher footing should be set below an imaginary line drawn up from the edge of the lower footing at 10 horizontal to 7 vertical. This practice will aid in limiting stress transfer from the higher footings to lower footings.

All footings exposed to the environment must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation to protect against frost damage. This frost protection would also be required if construction were undertaken during the winter months. All footings and foundations should be designed and constructed in accordance with the current Ontario Building Code.

With foundations designed as outlined above and as required by the Building Code, and with careful attention paid to construction detail, total and differential settlements should be well within normally tolerated limits of 25 and 20 millimetres respectively. However, as is typical in most new construction, 'cosmetic' cracking of foundation walls, etc. may occur within the first year of construction as a result of shrinkage, minor settlement, etc. Subsequent to repair, where warranted, additional cracking should be minimal.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this report and to allow changes to be made in the event that subsurface conditions differ from the conditions identified at the Borehole locations.

5. LATERAL EARTH PRESSURE

The lateral earth pressures on basement walls can be estimated on the basis of backfill [free draining granular material] unit weight, [γ], of 19.5 kN/m^3 [~124 pcf]. The coefficient of lateral earth pressure may be taken as, $k_o = 0.5$ in fill against rigid walls [at rest condition]. Any additional pressures due to surcharge loading, such as parked vehicles, floor slab loading, etc. must be included in the design. Where foundations are constructed as a 'waterproofed' section, they must also be designed to support the lateral hydrostatic pressure of a water level at ground surface.

6. SEISMIC DESIGN CONSIDERATIONS

The structure shall be designed according to Section 4.1.8 of the Ontario Building Code, Ontario Regulation 332/12. Assuming the proposed structure will be founded directly on the Dundas shale bedrock, based on the subsurface soil conditions encountered in this investigation the applicable Site Classification for the seismic design is Site Class B – Rock, based on the average soil characteristics for the site.

The seismic data from Supplementary Standard SB-1 of the Ontario Building Code for Mississauga (Port Credit) are as follows:

$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	PGA
0.280	0.150	0.065	0.021	0.150

7. EXCAVATIONS AND EXCAVATION SUPPORT CONSIDERATIONS

Excavations for the installation of foundations and underground services are anticipated to extend to depths of up to approximately 3 to 4 metres below the existing grade. Excavations through the sand soils encountered would be expected to remain stable at inclinations of up to 45 degrees to the horizontal. Excavations through the cohesive clayey silt encountered would be expected to remain stable at inclinations of up to 60 degrees to the horizontal, or steeper. Where excavations extend into the Dundas shale bedrock, they would be expected to remain stable at near vertical inclinations, the use of mechanical rock splitting equipment should be expected and the rate of excavation would be expected to be much slower. It is noted that foundations associated with the existing structures should be anticipated, and would expect to slow the rate of excavation. Notwithstanding the foregoing, however, all excavations must comply with the current Occupational Health and Safety Act and Regulations for Construction Projects. Excavation slopes steeper than those required in the Safety Act must be supported and a senior geotechnical engineer from this office should monitor the work.

It is anticipated that construction of the underground parking level will extend close to the property limits. Given the anticipated depths of construction, it may not be possible or feasible to advance the excavations with sloped faces as noted above, and the provision of excavation support such as soldier pile and timber lagging or caisson walls may be required. It is noted that the use of a timber lagging system is cautioned in sandy deposits, as there would be a potential for the sand to 'wash' through the lagging boards with resulting loss of ground and settlements behind the shoring.

A speciality contractor or shoring consultant should be consulted with respect to the design of such a shoring system, where required. For preliminary design purposes the shoring system should be designed on the basis of a retained soil unit weight of γ_{wet} = 19.5 kN/m^3 [~124 pcf], and a lateral earth pressure coefficient of $k_o = 0.5$ (at rest case) or $k_A = 0.3$ (active case). Caissons may be designed for end bearing using the values provided above within the Dundas shale bedrock.

The shoring system should be monitored during construction, and the contractor should have a contingency plan in place to be implemented should deflections of the shoring system exceed the tolerable limits. In addition, it is imperative that a pre-construction condition survey be conducted of the adjacent structures, roadways, etc. in order to document the existing conditions prior to the commencement of construction. This will allow for comparison and assessment in the event that disturbance due to construction activities is claimed.

The shoring can be designed as a cantilever, or supported either by anchors extending into the overburden soil or by rakers extending into the excavation, although from a contractor's point of view, tie-back anchors would be preferred, provided they can be installed to avoid adjacent foundations and underground utilities. The shoring must be monitored for movements, and a plan must be available before the excavations begin, to rectify the shoring system should movements become apparent. It is noted that significant movements of the shoring system may take place if conventional rakers are used to support the shoring system since compression of the members and the supporting footings must occur before the rakers can begin to carry load. In this regard, anchors are preferred since they allow pre-stressing of the shoring system to the design load even before the excavations reach their final grade. Alternatively, the rakers can be designed to allow jacking in the design load, and thus minimising movements. The design bearing capacity of the footings supporting inclined rakers should be limited to one half of the bearing capacities presented above for spread footing foundations.

As noted above the static groundwater level is anticipated to be approximately 3.0 metres below the existing ground surface, anticipated to be approximately coincident with, to just above the anticipated depths of construction. As such, infiltration of groundwater through more permeable seams, as well as surface runoff into open excavations, should be anticipated. It should be possible to adequately control groundwater infiltration for the short construction period using conventional construction dewatering methods, such as pumping from sumps in the base of the excavation. More sophisticated groundwater control methods may be required for deeper excavations. More groundwater control should be anticipated when connections are made to existing services, and where excavations extend into the Dundas shale bedrock. Surface water should be directed away from the excavations.

7. PERIMETER AND UNDERFLOOR DRAINAGE

With the provision of a single underground parking level, basement levels are anticipated to extend near to perhaps just below the estimated static groundwater level. As such, it may be prudent to design the foundations and basement slab to be water tight, making sure of suitable membrane systems beneath the slab and against the exterior of foundation walls. Where excavation shoring is utilized, this will likely require the use of foundation wall systems intended for 'blind side' or 'single face' application, where excavation shoring is provided. The system should also incorporate a water-stop component between the footing/grade beam/mat slab and foundation walls. This approach would avoid the requirement for permanent drainage and dewatering systems. The enclosed Drawing No. 2 shows a schematic of the typical requirements for water tight basement foundation construction. The foundation walls and floor slab/raft slab will also be required to be designed to resist the hydrostatic pressure, considering a high groundwater level of at the existing grade.

Alternatively, depending on the conditions experienced during construction, the shoring system utilised, or pending more detailed evaluation of the groundwater conditions, it may be feasible to adequately control groundwater infiltration against the foundations using permanent drainage systems. The foundation wall drainage and water proofing system will need to be installed against the excavation shoring where provided ('blind side' waterproofing). A variety of products are available for this application, such as Delta-Drain or TremDrain systems. Any system should include a drainage board layer and/or waterproofing membrane intended for a 'blind side' application. The system should also incorporate a water-stop component between the footing/grade beam/mat slab and foundation walls.

The volume of groundwater control required during construction should be monitored and used to assist in sizing the permanent drainage requirements. At a minimum it is recommended that the perimeter weeping tile consist of a 150-millimetre diameter perforated pipe, surrounded with 200 millimetres of 20-millimetre clear stone, with the stone in turn encased by a nonwoven filter fabric such as Mirafi 140N/Terrafix 270R or equivalent.

To address the potential for the build-up of groundwater beneath the basement floor slabs, in addition to underslab damp proofing measures recommended in the following section, under-floor drainage may be warranted. Under-floor drains may consist of 150-millimetre diameter perforated pipe, with a geofabric sock, placed in the clear stone beneath the floor slabs on nominal 4 to 6 metre centres. It is noted that the under-floor and perimeter drainage systems should have separate piping, i.e. piping from perimeter system does not connect to the under-floor system, in order to prevent surcharging of the under-floor system. They may outlet into a common sump-pit, though separate systems would be preferred. The enclosed Drawing No. 3 shows schematics of requirements for foundation construction with an underfloor drain system.

This office should examine the installation of the perimeter drains. Even a small break in the filtering materials could result in loss of fines into the drains with attendant performance difficulties, including settlements of the ground surface. The exterior grade around the structure should be sloped away from the structure to prevent the ponding of water against the foundation walls. The enclosed Drawing No. 2 shows a schematic of the typical requirements for perimeter and under floor drains for basement level construction.

Elevator pit excavations extending below the general basement floor level should also be designed to be water-tight. It is recommended that this water-tight design for the elevator pit be implemented regardless of the water proofing or drainage design adopted for the general basement level.

8. BASEMENT FLOOR SLABS

The lower level parking garage floor slab may be constructed using conventional slab-on-grade techniques on a prepared subgrade. The exposed subgrade should be evaluated in the presence of a representative of SOIL-MAT ENGINEERS. Any 'soft' spots delineated by this or by other means, should be sub-excavated and replaced with quality granular material compacted to 100 per cent of its standard Proctor maximum dry density [SPMDD].

As with all concrete floor slabs, there is a tendency for the floor slabs to crack. The slab thickness, concrete mix design, the amount of steel and/or fibre reinforcement and/or wire mesh placed into the concrete slab, if any, will therefore be a function of the owner's tolerance for cracks in, and movements of, the slabs-on-grade, etc. The 'saw cuts' in the concrete floors, for crack control, should extend to a minimum depth of 1/3 of the thickness of the slab.

A moisture barrier will be required under the floor slab such as the placement of at least 200 millimetres of well-compacted 20-millimetre clear crushed stone. At a minimum the moisture barrier material should contain no more than 10 per cent passing the No. 4 sieve. Where a 'non-damp' floor slab is required, as for instance under sheet vinyl floor coverings, etc., extra efforts will be required to damp proof the floor slab, as with the additional provision of a heavy 'poly' sheet, damp proofing sprays/membranes, drainage board products, etc. Where 'poly' sheets are used care should be taken to prevent puncturing and tearing and/or sufficiently heavy gauge sheeting specified. Alternatively, a proprietary product such as Delta-MS Underslab or WR Meadows membrane may be considered in lieu of the 'poly' sheets. Alternatively, the foundations and basement slab may be designed as water-tight making use of an appropriate continuous membrane product.

Curing of the slab-on-grade must be carefully specified to ensure that slab curl is minimised. This is especially critical during the hot summer months of the year when the surface of the slab tends to dry out quickly while high moisture conditions due to water trapped on top of the water proofing membrane at the saw cut joints and cracks, and at the edges of the slabs, maintains the underside of the slab in a moist condition.

It is also important that the concrete mix design provide a limiting water/cement ratio and total cement content, which will mitigate moisture related problems with low permeance floor coverings, such as debonding of vinyl and ceramic tile. It is equally important that excess free water not be added to the concrete during its placement as this could increase the potential for shrinkage cracking and curling of the slab.

As noted above it may be preferred to provide a water tight membrane system beneath the lower level parking garage slab of the mid-rise structure. The type of membrane system should be carefully selected and installed in accordance with the manufacturer specifications in order to achieve a permanent water tight condition. The installation of the water proofing system should be closely monitored to ensure it is continuous beneath the slab, with no breaks or gaps, and connects sufficiently to the foundation wall water proofing, to ensure that it will function per the manufacturer requirements.

9. PAVEMENT STRUCTURE DESIGN CONSIDERATIONS

All areas to be paved must be cleared of all topsoil and otherwise unsuitable materials and the exposed subgrade proof rolled with 3 to 4 passes of a loaded tandem-axle truck in the presence of a representative of SOIL-MAT ENGINEERS & CONSULTANTS LTD., immediately prior to the placement of the sub-base material. Any areas of distress revealed by this or other means should be subexcavated and replaced with suitable backfill material. Where the subgrade condition is poorer it may be necessary to implement more aggressive stabilisation methods, such as the use of coarse aggregate [50-millimetre clear stone, 'rip rap' stone, etc.] 'punched' into the soft areas. It may also be prudent to consider the provision of a heavy geofabric over the subgrade to act as a separator between the subgrade and granular base materials. It may be possible to reuse the granular materials present within the existing pavement structure, however this would be best assessed at the time of construction.

The need for subexcavations of softened subgrade materials will be reduced if construction is undertaken during dry periods of the year and careful attention is paid to the compaction operations. As noted above the on-site soils are sensitive to disturbance and moisture and may present difficulty for roadway construction during 'wet' periods of the year. Should pavement construction be undertaken during 'wet' periods of the year it should be anticipated that greater stabilisation efforts will be required and/or additional depth of OPSS Granular B, Type II sub-base course material may be required.

Good drainage provisions will optimise the long-term performance of the pavement structure. The subgrade must be properly crowned and shaped to promote drainage to the subdrain system. Subdrains should be installed to intercept excess subsurface water and to prevent softening of the subgrade material. Surface water should not be allowed to pond adjacent to the outer limits of the paved areas.

The most severe loading conditions on the subgrade typically occur during the course of construction, therefore precautionary measures may have to be taken to ensure that the subgrade is not unduly disturbed by construction traffic. SOIL-MAT should be given the opportunity to review the final pavement structure design and subdrain scheme prior to construction to ensure that they are consistent with the recommendations of this report.

The suggested pavement structures outlined in Table D are based on subgrade parameters estimated on the basis of visual and tactile examinations of the on-site soils and past experience. The outlined pavement structure may be expected to have an approximate ten-year life, assuming that regular maintenance is performed. Should a more detailed pavement structure design be required, site specific traffic information would be needed, together with detailed laboratory testing of the subgrade soils.

TABLE D
RECOMMENDED PAVEMENT STRUCTURES

LAYER DESCRIPTION	COMPACTION REQUIREMENTS	LIGHT DUTY SECTIONS	HEAVY DUTY [TRUCK ROUTE]
Asphaltic Concrete Wearing course OPSS HL 3 or HL 3A	92 per cent Marshall MRD	65 millimetres	40 millimetres
Binder Course OPSS HL 8	92 per cent Marshall MRD		65 millimetres
Base Course OPSS Granular A	100% SPMDD	150 millimetres	150 millimetres
Sub-base Course OPSS Granular B Type II	100% SPMDD	200 millimetres	350 millimetres

* Marshall MRD denotes Maximum Relative Density.

* SPMDD denotes Standard Proctor Maximum Dry Density, ASTM-D698.

Depending on the arrangement of light duty and heavy duty pavement sections, the transition between sections may present some difficulty for contractors. In this regard, consideration might be given to a slightly increased light duty pavement structure consisting of 50 millimetres of HL8 binder course and 40 millimetres of HL3 surface course asphaltic concrete. This structure will provide for a continuous depth of surface course asphalt allowing for ease of construction. As well such a pavement structure would have an improved performance over an increased design life. Such an arrangement of asphalt layers would also allow for future rehabilitation with a 'mill and pave' type operation.

Where asphalt pavement is to be constructed above the roof deck of a below grade parking level, the granular base layers recommended for the light duty pavement structure recommended above may be considered for both light duty and heavy duty areas. It is noted that in such cases the roof deck slab should be sufficiently sloped and/or provided with suitable subdrains, in order to promote rapid drainage of water from beneath the pavement. As well the roof slab should be provided with a suitable water proofing system.

To minimise segregation of the finished asphalt mat, the asphalt temperature must be maintained uniform throughout the mat during placement and compaction. All too often, significant temperature gradients exist in the delivered and placed asphalt with the cooler portions of the mat resisting compaction and presenting a honeycomb surface. As the spreader moves forward, a responsible member of the paving crew should monitor the pavement surface, to ensure a smooth uniform surface. The contractor can mitigate the surface segregation by 'back-casting' or scattering shovels of the full mix material over the segregated areas and raking out the coarse particles during compaction operations. Of course, the above assumes that the asphalt mix is sufficiently hot to allow the 'back-casting' to be performed.

10. GENERAL COMMENTS

The comments provided in this document are intended only for the guidance of the design team. The subsoil descriptions and borehole information are only intended to describe conditions at the borehole locations. Contractors placing bids or undertaking this project should carry out due diligence in order to verify the results of this investigation and to determine how the subsurface conditions will affect their operations.

We trust that this geotechnical report is sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

LICENSED PROFESSIONAL ENGINEER
May 8/19
S. K. RICHARDSON
100179716
PROVINCE OF ONTARIO

Kyle Richardson, P. Eng.
Project Engineer

Ian Shaw, P.Eng., QP_{ESA}
Senior Engineer

Enclosures: Drawing No.1, Borehole Location Plan
Log of Borehole Nos. 1 and 2
Drawing No. 2, Typical Design Requirements, Water Tight Basement Foundations
Drawing No. 3, Typical Design Requirements, Drainage and Backfill for Exterior Walls with Underfloor Drains

Distribution: Oakhill Environmental Ltd. [1, plus pdf]

LEGEND

BH# Borehole Location

TBM Temporary Benchmark
Water valve cover.
Assumed Elevation of 100.00 m.

NOTES

1. This drawing should be read in conjunction with Soil-Mat Engineers & Consultants Ltd. Report No. SM 190138-G.

2. Borehole locations are approximate.

SOIL-MAT
ENGINEERS & CONSULTANTS LTD.

Proposed Condominium Development
3085 Hurontario Street
Mississauga, Ontario

Borehole Location Plan

Project No. SM 190138-G

Date: April 2019

Drawn: ZRV | Checked: KR

SM 190138-G Borehole Location Plan

Drawing No. 1

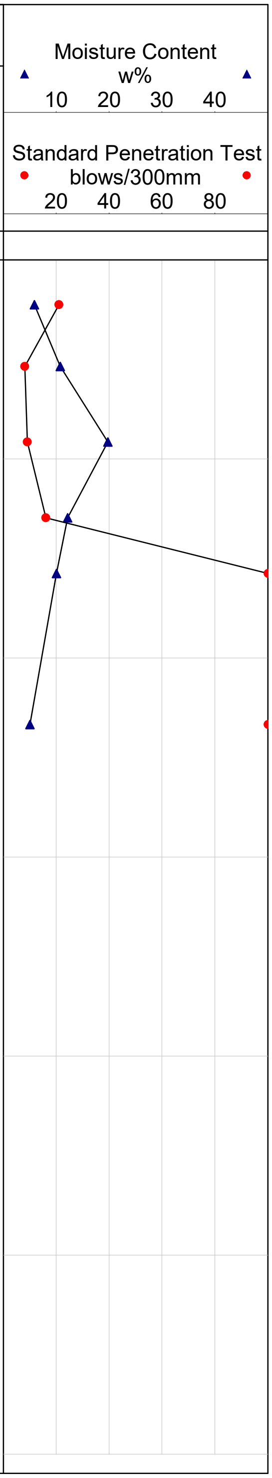
Log of Borehole No. 1

Project No: SM 190138-G
Project: Proposed Condominium Building
Location: 3085 Hurontario Street, Mississauga
Client: Oakhill Environmental Inc.

Project Manager: Kyle Richardson
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4826460
E: 611511

Elevation (m)	Description
100.03	Ground Surface
99.73	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 200 millimetres of compact granular base.
97.80	**Sand** Brown, medium in gradation, trace gravel, occasional organics in upper level, loose.
96.70	**Clayey Silt** Grey, trace gravel, very stiff.
92.10	**Dundas Shale** Grey with occasional harder limestone layers, highly weathered in upper levels, becoming more sound with depth, hard.
	End of Borehole

SAMPLE						
Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
SS	1	9,10,11,7	21	▨		
SS	2	3,4,4,9	8	▨		
SS	3	5,5,4,6	9	▨		
SS	4	4,7,9,12	16	▨	>4.5	
SS	5	14,50/5"	100	▨		
SS	6	50/4"	100	▨		
NQ	7	RQD 29.4%		▨		
NQ	8	RQD 35.7%		▨		



NOTES:

1. Borehole was advanced using hollow stem auger equipment on April 8, 2019 to auger refusal at a depth of 5.2 metres, then the bedrock cored to a depth of approximately 7.9 metres using Nq diamond barrel equipment.

2. Borehole was backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Hollow Stem Augers
Drill Date: April 8, 2019
Hole Size: 200 millimetres
Drilling Contractor: Geo-Environmental

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

Datum: Temporary Benchmark
Field Logged by: ZRV
Checked by: KR
Sheet: 1 of 1

Log of Borehole No. 2

Soil-Mat

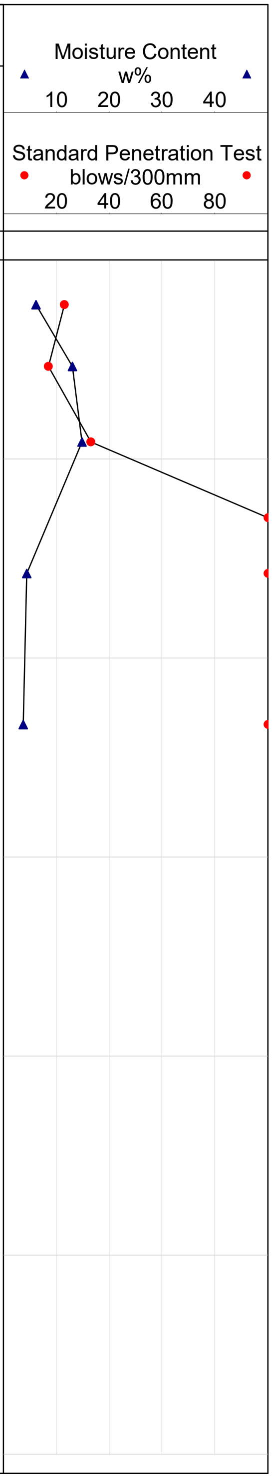
Project No: SM 190138-G
Project: Proposed Condominium Building
Location: 3085 Hurontario Street, Mississauga
Client: Oakhill Environmental Inc.

Project Manager: Kyle Richardson
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4826436
E: 611503

Elevation (m)	Description
99.79	Ground Surface
99.49	**Pavement Structure** Approximately 150 millimetres of asphaltic concrete over 150 millimetres of compact granular base.
98.00	**Sand** Brown, medium in gradation, trace gravel, occasional organics in upper level, compact.
97.30	**Clayey Silt** Grey, trace gravel, hard.
	Dundas Shale Grey with occasional harder limestone layers, highly weathered in upper levels, becoming more sound with depth, hard.
95.10	End of Borehole

SAMPLE

Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
SS	1	12,12,11,9	23	▩		
SS	2	3,5,12,19	17	▩		
SS	3	12,22,11,13	33	▩		
SS	4	11,50/4"	100			
SS	5	50/5"	100	▩		
SS	6	50/3"	100	▩		



NOTES:

1. Borehole was advanced using hollow stem auger equipment on April 8, 2019 to auger refusal on assumed bedrock at a depth of approximately 4.6 metres.

2. Borehole was backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

April 24, 2019 - 3.1 metres
May 7, 2019 - 3.0 metres

Drill Method: Hollow Stem Augers
Drill Date: April 8, 2019
Hole Size: 200 millimetres
Drilling Contractor: Geo-Environmental

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Datum: Temporary Benchmark
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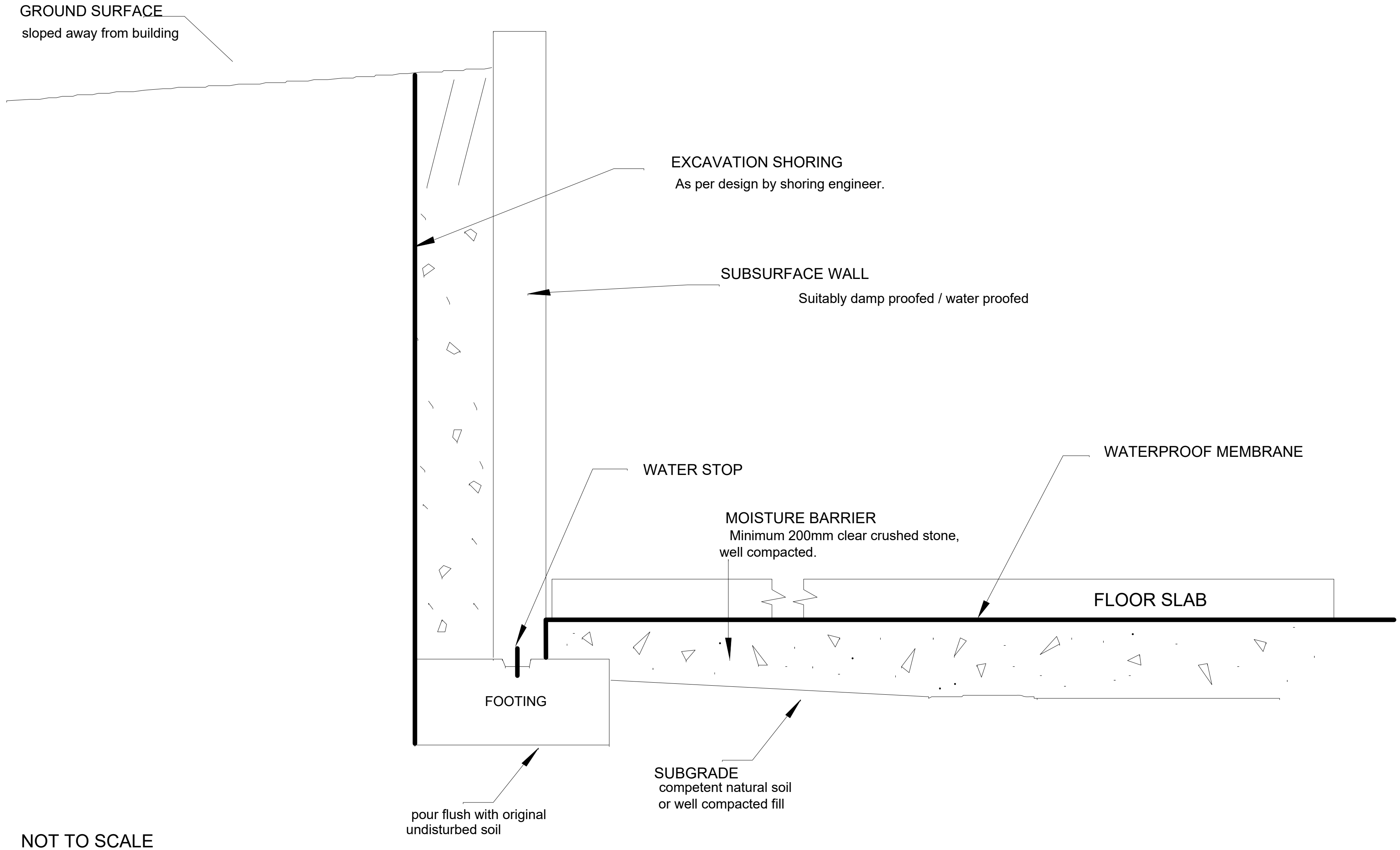
Sheet: 1 of 1



NOT TO SCALE

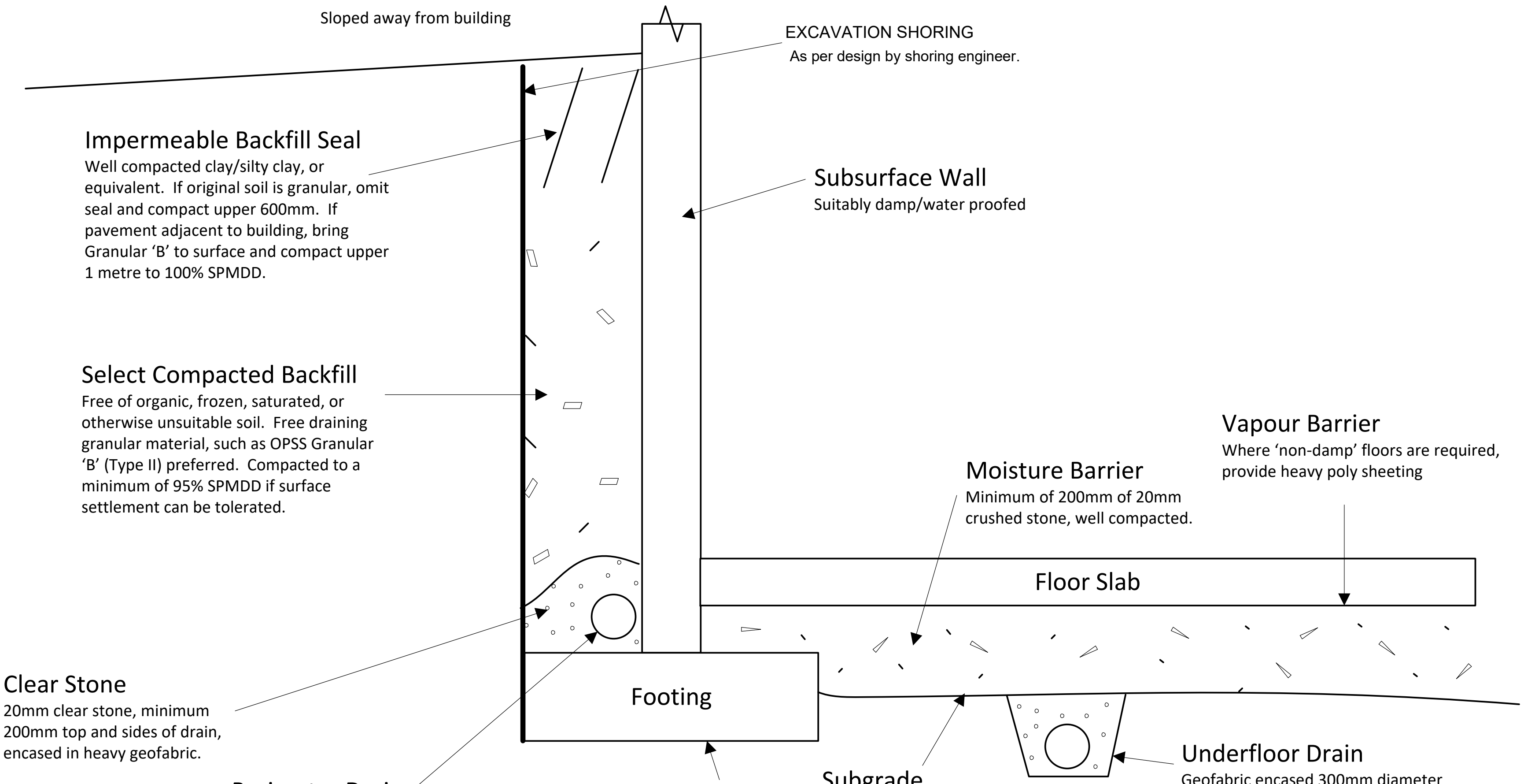
Soil-Mat

Soil-Mat Engineers & Consultants Ltd.

Project No.:	SM 190138-G
Date:	May 2019

Typical Design Requirements
Water Tight Basement Foundations

Drawing No. 2



Ground Surface
Sloped away from building

EXCAVATION SHORING
As per design by shoring engineer.

Impermeable Backfill Seal
Well compacted clay/silty clay, or equivalent. If original soil is granular, omit seal and compact upper 600mm. If pavement adjacent to building, bring Granular 'B' to surface and compact upper 1 metre to 100% SPMDD.

Subsurface Wall
Suitably damp/water proofed

Select Compacted Backfill
Free of organic, frozen, saturated, or otherwise unsuitable soil. Free draining granular material, such as OPSS Granular 'B' (Type II) preferred. Compacted to a minimum of 95% SPMDD if surface settlement can be tolerated.

Vapour Barrier
Where 'non-damp' floors are required, provide heavy poly sheeting

Moisture Barrier
Minimum of 200mm of 20mm crushed stone, well compacted.

Clear Stone
20mm clear stone, minimum 200mm top and sides of drain, encased in heavy geofabric.

Perimeter Drain
150mm diameter weeping tile or pipe equivalent, leading to positive sump or outlet. Invert at least 150mm below underside of floor slab.

Pour flush with original undisturbed soil.

Subgrade
Competent native soil or well compacted fill.

Underfloor Drain
Geofabric encased 300mm diameter perforated pipe or equivalent, leading to a positive sump or outlet. Invert at least 300mm below underside of floor slab. Pipe placed in parallel rows on nominal 4-6m centres.

NOT TO SCALE

Soil-Mat

Soil-Mat Engineers & Consultants Ltd.

Project No.: SM 190138-G
Date: May 2019

Typical Design Requirements
Drainage and Backfill for Basement Walls

Drawing No. 3

SOIL-MAT ENGINEERS & CONSULTANTS LTD.

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Milton: PO Box 40012 Derry Heights PO L9T 7W4 T: 800.243.1922

PROJECT NO.: SM 200167-G April 24, 2020

EQUITY BUILDERS
53 Village Centre Place
Mississauga, Ontario
L4Z 1V9

Attention: Ash Singh
President and Founder

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED CONDOMINIUM DEVELOPMENT
3085 HURONTARIO STREET
MISSISSAUGA, ONTARIO

Dear Mr. Singh,

Further to your authorisation, SOIL-MAT ENGINEERS & CONSULTANTS LTD. has completed the fieldwork, laboratory testing, and report preparation in connection with the above noted project. The scope of work was completed in general accordance with our proposal P8559 dated January 24, 2020, and should be read in conjunction with our previous Geotechnical Investigation on the subject property [Report No. SM 190138-G, dated May 8, 2019]. Our comments and recommendations based on our findings at the supplemental borehole location are presented in the following paragraphs.

1. INTRODUCTION

We understand that the number of underground levels has potentially increased from one to up to four since the completion of the original geotechnical investigation. The purpose of this supplemental geotechnical investigation work was to further assess the groundwater and condition of the bedrock at greater depths, and to provide our comments and recommendations with respect to the design and construction of foundations for the proposed development, from a geotechnical point of view.

2. PROCEDURE AND SUBSURFACE CONDITIONS

A single borehole identified as Borehole No. 101 was advanced at the location illustrated in the attached Drawing No. 1A, Borehole Location Plan. It is noted that given the presence of existing buildings and parking structures, the relatively small on grade parking lot in the central portion of the site was the only area accessible with drill equipment at the time of our fieldwork. The borehole was advanced using continuous flight power auger equipment under the direction of a representative of SOIL-MAT ENGINEERS to practical auger refusal on Dundas shale bedrock at a depth of approximately 2.8 metres below the existing pavement surface. The borehole was then advanced using Nq diamond barrel coring equipment from approximately 3.3 to 13.9 metres. Recovered core samples of the bedrock were preserved and returned to the SOIL-MAT laboratory for testing including Rock Quality Designation [RQD] and unconfined compressive strength testing. The results of this testing is presented in the following text.

Upon completion of drilling, a groundwater monitoring well was installed at the borehole location to allow for future measurements of the static groundwater level. The monitoring well consisted of 50-millimetre diameter PVC pipe, screened in the lower 9.1 metres. The well was encased in well filter sand up to approximately 0.3 metres above the screened portion, then with a bentonite ‘hole plug’ up to the surface, and fitted with a protective steel monument casing.

The elevation of the ground surface at the borehole locations was determined relative to a geodetic benchmark, described as the door sill located on the north face of the existing building located at 3085 Hurontario Street West. This benchmark was noted to have a geodetic elevation of 116.40 metres as per the Site Servicing and Grading Plan drawing by McConnell Maughan Limited Plan 313E-2., dated July 1986, provided to our office by our client.

The borehole was advanced through the pavement structure of the existing on grade parking lot, which was found to consist of approximately 100 millimetres of asphaltic concrete overlying 200 millimetres of compact granular base. Native sand was encountered beneath the pavement structure. The sand was brown in colour, contained trace gravel, and was generally in a loose state. Native sand was proven to a depth of approximately 2.1 metres were native clayey silt overburden soils were encountered. The native cohesive soils were noted to be grey in colour, contained trace gravel, and were generally very stiff in consistency. The native clayey silt was proven to a depth of approximately 2.8 metres below the existing grade.

Dundas Shale Bedrock

Dundas shale bedrock was encountered beneath the cohesive clayey silt, at a depth of approximately 2.8 metres. It is noted that the upper levels of the Dundas shale bedrock are highly weathered, exhibiting characteristics of a very stiff to hard cohesive soil, such that the upper about 0.5 metres was penetrated by auger equipment. As such, the transition from the very stiff to hard cohesive overburden clayey silt to weathered Dundas shale bedrock is somewhat indistinct. The depth and elevation at which bedrock was encountered at the borehole locations, including those boreholes previously advanced, have been summarised as follows:

TABLE A – BEDROCK DEPTH/ELEVATION

	Surface Elevation (m)	Weathered Bedrock Depth (m)	Bedrock Elevation (m)
Borehole No. 1	116.39	3.3	113.10
Borehole No. 2	116.15	2.5	113.70
Borehole No. 101	116.23	2.8	113.40

As noted above, approximately 10.6 metres of the bedrock was cored in Borehole No. 101 from approximately 3.3 to 13.9 metres below the existing grade, using Nq diamond barrel coring equipment. The cores were noted to yield recoveries of approximately 93 to 100 per cent, with a Rock Quality Designation [RQD] of approximately 24% to 79%. Unconfined compressive strength testing on selected portions of the recovered samples yielded compressive strengths of approximately 11.8 to 69.3 MPa, with an average of 24.5 MPa. This is consistent with a highly weathered, weak rock of poor to moderate quality. Attempts to perform unconfined compressive strength testing on recovered core samples from the upper 5 metres of the bedrock were unsuccessful due to the highly weathered condition of the bedrock. The results of our analysis of the bedrock including the unconfined compressive strength tests, where possible, have been summarised as follows:

TABLE B – SUMMARY OF BEDROCK CORING

Depth of Core (m)	Elevation of Core (m)	Recovery (%)	Rock Quality Designation (RQD) (%)	Tested Core Sample		Unconfined Compressive Strength (MPa)
				Depth (m)	Elevation (m)	
Borehole No. 101						
3.3 to 4.8	112.9 to 111.4	100	0	NA	NA	NA
4.8 to 6.3	111.4 to 109.9	100	64	NA	NA	NA
6.3 to 7.8	109.9 to 108.4	100	79	NA	NA	NA

Depth of Core (m)	Elevation of Core (m)	Recovery (%)	Rock Quality Designation (RQD) (%)	Tested Core Sample		Unconfined Compressive Strength (MPa)
				Depth (m)	Elevation (m)	
7.8 to 9.3	108.4 to 106.9	97	63	8.5 8.9	107.7 107.3	13.8 13.5
9.3 to 10.9	106.9 to 105.3	93	44	9.5 9.8	106.7 106.4	11.8 14.2
10.9 to 12.2	105.3 to 104.0	100	24	11.1	105.1	69.3
12.2 to 13.9	104.0 to 102.3	100	57	12.9 13.1	103.3 103.1	56.3 12.4
Borehole No. 1						
5.2 to 6.4	111.2 to 110.0	100	29	NA		NA
6.4 to 7.9	110.0 to 108.5	100	36	7.1 7.8	109.3 108.6	15.3 18.9

Groundwater Observations

Borehole No. 101 was noted to be 'dry' upon completion of drilling. It is noted that insufficient time would have passed for the static groundwater level to stabilise in the open boreholes. As noted above, a monitoring well was installed within the borehole to allow for future measurements of the groundwater level. The water levels measured at this location have been summarised as follows:

TABLE C – SUMMARY OF GROUNDWATER MEASUREMENTS

Date	Groundwater Depth (m)	Groundwater Elevation (m)
Borehole No. 101 – Surface Elevation 116.23 m		
March 27, 2020	4.6	111.6
April 17, 2020	4.5	111.7
Borehole No. 2 – Surface Elevation 116.15 m		
April 24, 2019	3.1	113.1
May 7, 2019	3.0	113.2
April 17, 2020	3.1	113.1

Based on our observations during drilling and measurements of the groundwater level taken from the monitoring well and the existing monitoring wells installed in our prior completed investigation, the static groundwater level is estimated at a depth of approximately 3 to 3.6 metres below the existing grade, at an elevation of approximately 113.2 metres, perhaps deeper, and would be expected to fluctuate seasonally. As the proposed depths of construction are well below the depths of the estimated static groundwater level, a more detailed hydrogeological dewatering assessment is necessary to further assess the groundwater condition and implication of excavations and basement levels below the groundwater level.

3. FOUNDATION CONSIDERATIONS

With up to four underground levels, it is anticipated that the newly proposed founding depth will be on the order of 10 to 12 metres, well within the depths of the Dundas shale bedrock. Foundations founded at this depth may be designed considering a bearing capacity of 2,000 kPa [~40,000 psf] in both SLS and ULS, based on the compressive strength testing of the recovered bedrock cores detailed above. The footing beds must be hand cleaned of any loose or disturbed material, along with any ponded water, immediately prior to the placement of concrete.

It is noted that the SLS value represents the Serviceability Limit State, which is governed by the tolerable deflection [settlement] based on the proposed building type, using unfactored load combinations. The ULS value represents the Ultimate Limit State and is intended to reflect an upper limit of the available bearing capacity of the founding soils in terms of geotechnical design, using factored load combinations. There is no direct relationship between ULS and SLS; rather they are a function of the soil type and the tolerable deflections for serviceability, respectively. Evidently, the bearing capacity values would be lower for very settlement sensitive structure and larger for more flexible buildings. It is noted that the SLS and ULS bearing capacities are equivalent within the competent Dundas shale bedrock, as in order for the serviceability limit states to be realised, ultimate failure of the bedrock would have to occur.

In areas where it will be necessary to provide adjacent footings at different founding elevations, the lower footing should be constructed before the higher footing is constructed, if possible, and the higher footing should be set below an imaginary line drawn up from the edge of the lower footing at 10 horizontal to 7 vertical. This practice will aid in limiting stress transfer from the higher footings to lower footings.

All footings exposed to the environment must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation to protect against frost damage. This frost protection would also be required if construction were undertaken during the winter months. All footings and foundations should be designed and constructed in accordance with the current Ontario Building Code.

With foundations designed as outlined above and as required by the Building Code, and with careful attention paid to construction detail, total and differential settlements should be well within normally tolerated limits of 25 and 20 millimetres respectively. However, as is typical in most new construction, 'cosmetic' cracking of foundation walls, etc. may occur within the first year of construction as a result of shrinkage, minor settlement, etc. Subsequent to repair, where warranted, additional cracking should be minimal.

It is imperative that a soils engineer be retained from this office to provide geotechnical engineering services during the excavation and foundation construction phases of the project. This is to observe compliance with the design concepts and recommendations of this report and to allow changes to be made in the event that subsurface conditions differ from the conditions identified at the Borehole locations.

4. EXCAVATIONS

As noted above excavations for the installation of the proposed basement levels are anticipated to extend to depths of up to approximately 10 to 12 metres below the existing grade, through the sandy overburden soils and into the Dundas shale bedrock. Excavations through the overburden soils and upper weathered layers of the Dundas shale bedrock may be readily advanced using heavy hydraulic excavator equipped with ‘rock’ teeth. Excavations extending into the competent Dundas shale bedrock will require the use the mechanical rock splitting equipment, and the rate of excavation would be expected to slow significantly.

Where adequate space is available excavations through the overburden soils and upper weathered levels of the Dundas Shale may be advanced as open cuts. Excavations through the sand overburden soils encountered can be expected to remain stable at inclinations of up to 45 degrees to the horizontal. However, where wet seams or present or due to surface runoff the silty sand soils will have a tendency to locally ‘slump-in’ to slopes as flat as 3 horizontal to 1 vertical, or flatter. In this regard it may be necessary to provide some level of temporary protection or support depending on the proximity of excavations to the property line or adjacent above or below grade structures. At a minimum the excavation faces in soil should be covered with heavy construction tarps, secured at the top and bottom, to protect the exposed faces from damage due to rainfall and surface runoff. This office should be provided the opportunity to review the excavation shoring requirements and details, once established, in order to confirm they are consistent with the recommendations of this report and provide recommendations for additional measures where required.

A specialty contractor or shoring consultant should be consulted with respect to the design of such a shoring system, where required, based on the conditions encounters. For preliminary design purposes the shoring system should be designed on the basis of a retained soil unit weight of γ_{wet} = 19.5 kN/m^3 [~124 pcf] for the overburden sand, and a lateral earth pressure coefficient of k_o = 0.5 (at rest case) or k_A = 0.3 (active case).

Excavations into the Dundas shale bedrock, below the highly weathered upper levels, can be expected to remain stable at near vertical inclinations. Where workers are required to enter the excavations the sides should be thoroughly scaled of all loose rock debris and routinely evaluated by a senior engineer from this office. Consideration should be given to covering the excavation faces with heavy construction tarps or geotextile net, secured at top and bottom, to direct any lose rock debris that may come loose safely to the bottom of the excavation. Notwithstanding the foregoing, however, all excavations must comply with the current Occupational Health and Safety Act and Regulations for Construction Projects.

It is strongly recommended that pre-construction surveys of adjacent structures and underground services be undertaken prior to the start of construction in order to document their existing condition. This will allow for comparison and assessment in the event that disturbance due to construction activities is claimed.

Excavations to accommodate the proposed basement levels are expected to extend as much as 8 metres or more below the estimated static groundwater level, as measured at the monitoring well locations. Significant volumes of infiltration through permeable seams in the Dundas shale bedrock should be anticipated. As noted above, a detailed hydrogeological dewatering assessment would be required to better assess the rate of infiltration and to allow for the development of the dewatering scheme, and assess the potential for requirement of a Permit to Take Water during construction.

5. GENERAL COMMENTS

The comments provided in this document are intended only for the guidance of the design team. The subsoil descriptions and borehole information are only intended to describe conditions at the borehole locations. Contractors placing bids or undertaking this project should carry out due diligence in order to verify the results of this investigation and to determine how the subsurface conditions will affect their operations.

We trust that this geotechnical report is sufficient for your present requirements. Should you require any additional information or clarification as to the contents of this document, please do not hesitate to contact the undersigned.

Yours very truly,
SOIL-MAT ENGINEERS & CONSULTANTS LTD.

Scott Wylie, B.Eng., EIT

LICENSED PROFESSIONAL ENGINEER
Ap-24/20
S. K. RICHARDSON
100179716
PROVINCE OF ONTARIO

Kyle Richardson, P. Eng.
Project Engineer

Stephen R. Sears, B. Eng. Mgmt., P. Eng., QP$_{ESA}$
Senior Engineer

Enclosures:	Drawing No.1A, Borehole Location Plan Log of Borehole Nos. 1, 2, and 101
Distribution:	Equity Builders [1, plus pdf]

LEGEND

- Borehole Location — BH#
- Benchmark — TBM — Door sill. Assumed elevation of 116.40 metres

NOTES

1. This drawing should be read in conjunction with Soil-Mat Engineers & Consultants Ltd. Geotechnical Report SM 200167-G.

2. Borehole locations are approximate.

SOIL-MAT

ENGINEERS & CONSULTANTS LTD.

Geotechnical Investigation
Proposed Condominium
Development
3085 Hurontario Street
Mississauga, Ontario

Borehole Location Plan

Project No. SM 200167-G

Date: April 2020

Drawn: SW | Checked: KR

SM 200167-G Borehole Location Plan

Drawing No. 1

Log of Borehole No. 1

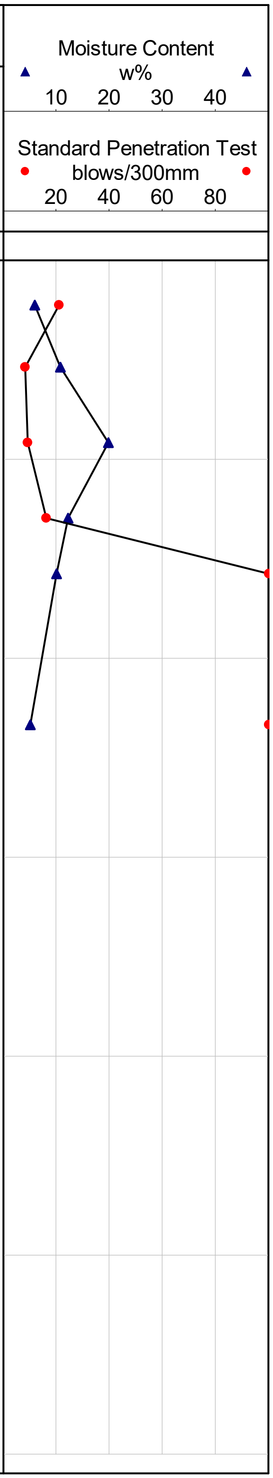
Project No: SM 190138-G
Project: Proposed Condominium Building
Location: 3085 Hurontario Street, Mississauga
Client: Oakhill Environmental Inc.

Project Manager: Kyle Richardson
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4826460
E: 611511

Soil-Mat

Depth	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
ft m	116.39		Ground Surface								
	116.09		**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 200 millimetres of compact granular base.		SS	1	9,10,11,7	21			
			Sand Brown, medium in gradation, trace gravel, occasional organics in upper level, loose.		SS	2	3,4,4,9	8			
	114.20				SS	3	5,5,4,6	9			
			Clayey Silt Grey, trace gravel, very stiff.		SS	4	4,7,9,12	16		>4.5	
	113.10				SS	5	14,50/5"	100			
			Dundas Shale Grey with occasional harder limestone layers, highly weathered in upper levels, becoming more sound with depth, hard.			6	50/4"	100			
					NQ	7	RQD 29.4%				
	108.50				NQ	8	RQD 35.7%				
			End of Borehole								

Depth scale (ft): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39
Depth scale (m): 0 1 2 3 4 5 6 7 8 9 10 11



NOTES:

1. Borehole was advanced using hollow stem auger equipment on April 8, 2019 to auger refusal at a depth of 5.2 metres, then the bedrock cored to a depth of approximately 7.9 metres using Nq diamond barrel equipment.

2. Borehole was backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

Drill Method: Hollow Stem Augers
Drill Date: April 8, 2019
Hole Size: 200 millimetres
Drilling Contractor: Geo-Environmental

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
E: info@soil-mat.ca

Datum: Benchmark
Field Logged by: ZRV
Checked by: KR
Sheet: 1 of 1

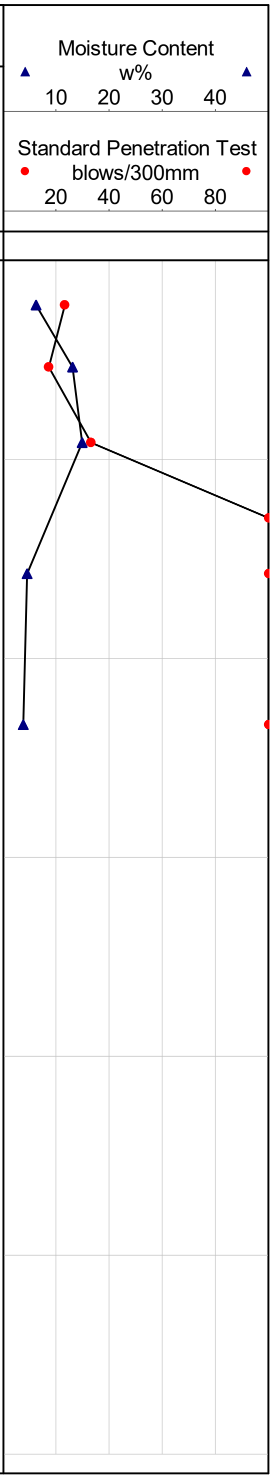
Log of Borehole No. 2

Project No: SM 190138-G
Project: Proposed Condominium Building
Location: 3085 Hurontario Street, Mississauga
Client: Oakhill Environmental Inc.

Project Manager: Kyle Richardson
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4826436
E: 611503

Elevation (m)	Description	Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)
116.15	Ground Surface							
115.85	**Pavement Structure** Approximately 150 millimetres of asphaltic concrete over 150 millimetres of compact granular base.	SS	1	12,12,11,9	23			
114.40	**Sand** Brown, medium in gradation, trace gravel, occasional organics in upper level, compact.	SS	2	3,5,12,19	17			
113.70	**Clayey Silt** Grey, trace gravel, hard.	SS	3	12,22,11,13	33			
111.50	**Dundas Shale** Grey with occasional harder limestone layers, highly weathered in upper levels, becoming more sound with depth, hard.	SS	4	11,50/4"	100			
		SS	5	50/5"	100			
			6	50/3"	100			

Depth: 0–39 ft; 0–11 m



End of Borehole

NOTES:

1. Borehole was advanced using hollow stem auger equipment on April 8, 2019 to auger refusal on assumed bedrock at a depth of approximately 4.6 metres.

2. Borehole was backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free groundwater level readings have been measured:

April 24, 2019 - 3.1 metres

May 7, 2019 - 3.0 metres

April 17, 2020 - 3.1 metres

Drill Method: Hollow Stem Augers
Drill Date: April 8, 2019
Hole Size: 200 millimetres
Drilling Contractor: Geo-Environmental

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Datum: Temporary Benchmark
Field Logged by: ZRV
Checked by: KR
Sheet: 1 of 1

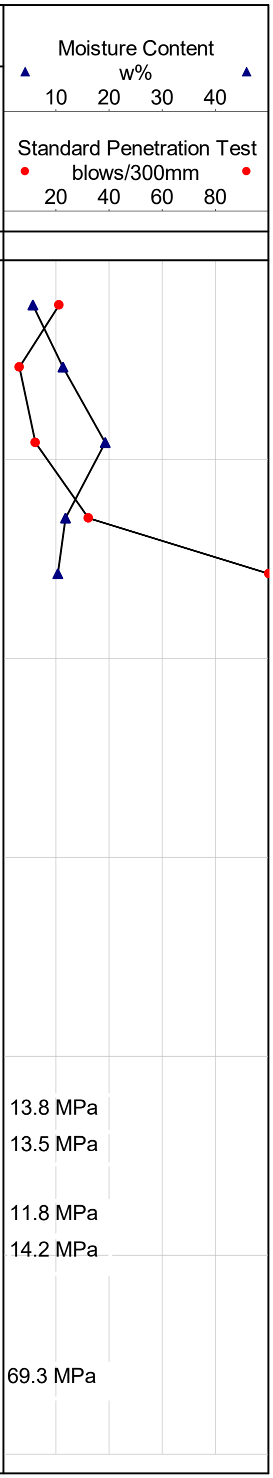
Log of Borehole No. 101

Project No: SM 190138-G
Project: Proposed Condominium Building
Location: 3085 Hurontario Street, Mississauga
Client: Oakhill Environmental Inc.

Project Manager: Kyle Richardson
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4826448
E: 611500

Elevation (m)	Description
116.23	Ground Surface
115.93	**Pavement Structure** Approximately 100 millimetres of asphaltic concrete over 200 millimetres of compact granular base.
114.10	**Sand** Brown, medium in gradation, trace gravel, loose to compact.
113.40	**Clayey Silt** Grey, trace gravel, very stiff.
	Dundas Shale Grey with occasional harder limestone layers, highly weathered in upper levels, becoming more sound with depth, hard.

SAMPLE Type	Number	Blow Counts	Blows/300mm	PP (kgf/cm2)
SS	1	12,11,10,9	21	
SS	2	5,4,2,2	6	
SS	3	4,5,7,9	12	
SS	4	6,10,22,50/3"	32	>4.5
SS	5	50/3"	100	
NQ	7	RQD 0%		
NQ	8	RQD 64.2%		
NQ	9	RQD 78.8%		
NQ	10	RQD 62.9%		
NQ	11	RQD 44.2%		
NQ	12	RQD 23.6%		



Drill Method: Hollow Stem Augers
Drill Date: March 12, 2020
Hole Size: 200 millimetres
Drilling Contractor: Davis Drilling

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Datum: Temporary Benchmark
Field Logged by: SW
Checked by: KR
Sheet: 1 of 2

Log of Borehole No. 101

Project No: SM 190138-G
Project: Proposed Condominium Building
Location: 3085 Hurontario Street, Mississauga
Client: Oakhill Environmental Inc.

Project Manager: Kyle Richardson
Borehole Location: See Drawing No. 1
UTM Coordinates - N: 4826448
E: 611500

Soil-Mat

Depth	Elevation (m)	Symbol	Description	Well Data	SAMPLE Type	Number	Blow Counts	Blows/300mm	Recovery	PP (kgf/cm2)	U.Wt.(kN/m3)	Moisture Content w% (10 20 30 40) / Standard Penetration Test blows/300mm (20 40 60 80)
40–45	102.40				NQ	13	RQD 56.7%					56.3 MPa 12.4 MPa
46–79			End of Borehole									

Depth: 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74 75 76 77 78 79

NOTES:

1. Borehole was advanced using hollow stem auger equipment on March 12, 2020 to auger refusal at a depth of 3.0 metres, then the bedrock cored to a depth of approximately 13.8 metres using Nq diamond barrel equipment.

2. Borehole was backfilled as per Ontario Regulation 903.

3. Soil samples will be discarded after 3 months unless otherwise directed by our client.

4. A monitoring well was installed. The following free ground water level readings have been measured:

March 27, 2020 - 4.6 metres below the existing ground surface

April 17, 2020 - 4.5 metres below the existing ground surface

Drill Method: Hollow Stem Augers
Drill Date: March 12, 2020
Hole Size: 200 millimetres
Drilling Contractor: Davis Drilling

Soil-Mat Engineers & Consultants Ltd.
130 Lancing Drive, Hamilton, ON L8W 3A1
T: 905.318.7440 F: 905.318.7455
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Datum: Temporary Benchmark
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Sheet: 2 of 2

APPENDIX F

Construction Dewatering Assessment Report