FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT

55 Port Street East

City of Mississauga (OZ/OPA 18 7)
Region of Peel (TBD)

Prepared for
FRAM + SLOKKER

Project #: 17-548W

1st Submission (Zoning) - February 2018
2nd Submission – December 2018

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1. INTRODUCTION

1.1. Background

Urbantech West has been retained to prepare a Functional Servicing Report / Stormwater Management Report in support of an official plan and zoning by-law amendment application for 55 Port Street East (hereafter referred to as the "subject lands" or "site"). The site is located southeast corner of Port Street East and Helene Street South in the City of Mississauga. The legal description of this property is Block 9 and Block 10, Plan 43M-1463.

Refer to **Figure 1** for the **Site Location Plan**

This report reviews offsite servicing capacities and provides functional servicing design and stormwater management information for the proposed development. The proposed site grading, site servicing and stormwater management designs are in accordance with accepted engineering practices, as well as, both City of Mississauga and Region of peel standards and specifications.

1.2. Subject Site

The site is approximately 0.23 ha in size and is currently occupied by an existing commercial building and associated parking. The site is bounded by Port Street East to the north, Lake Ontario/Waterfront Trail to the south, existing residential development to the east and Helene Street South to the west. The site is a part of the Credit Valley Conservation Watershed, within the Norval to Port Credit subwatershed.

A geotechnical investigation was prepared by Terraprobe Inc. (August 2018). In the north end of the site, it was determined that there is approximately 0.08-0.09 m asphalt concrete underlain by 0.12 m to 0.67 m granular base. In the south end of the site it was determined that there is approximately 0.10 m of topsoil underlain with 0.80 m to 2.30 m of fill material consisting of clayey silt to sand/silty sand. The groundwater levels range from 3 m to 6.8 m within the site or between an elevation of 71.5 to 74.7 respectively. It is likely that the stabilized ground water level elevation is similar to the water level in Lake Ontario.

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

1.3. Proposed Development

The proposed works include redeveloping the subject lands with a 10 storey, 34 unit residential development with underground parking areas, associated water, storm and sanitary servicing. Vehicle access to the underground parking is proposed at Port Street East through the loading bay.

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55 PORT STREET EAST

FIGURE 1
SITE LOCATION PLAN

PROJECT No.:	17-548W
DATE:	NOVEMBER 2017
SCALE:	1:1250
FIGURE No.:	1

File: F:\PROJECTS\17-548 (FRAM AND SLOKKER - 55 PORT STREET EAST)\DRAWINGS\00_FUNCTIONAL\05_FD_FIGURES\SITE LOCATION PLAN.DWG

PLOT DATE: 12/6/2018 11:43:47 AM

2. GRADING

The future grades required to service the site will generally be influenced by boundary conditions and matching existing grades on the north, south, east and west sides of the site. The site grading design will take into consideration the following requirements and constraints:

- Conform to City of Mississauga's design criteria.
- Minimize cut and fill operations and work towards a balanced site.
- Match existing boundary conditions.
- Provide overland flow conveyance for major storm conditions.
- Reduce the number of gravity servicing outlets.
- Reduce or eliminate the need for retaining walls.
- Provide suitable cover on proposed servicing.
- Achieve stormwater management and environmental objectives required for the site.

The grading has been designed to match the existing perimeter to minimize disturbance to the existing boundaries. Please refer to **Drawing GSP-1** for **Grading Plan**.

3. STORM SERVICING AND STORMWATER MANAGEMENT

3.1. Existing Storm Servicing

Underground services on Helene St. are sized to convey the 10-year storm event. Flows within the site are captured at three internal low points; one at the north end of the lot and two at the south end of the lot and are conveyed to a 450mm diameter storm sewer within Helene Street South. This sewer extends beneath the waterfront trail and discharges into Lake Ontario via a headwall.

The existing 10-year storm design sheet is included in **Appendix B.** The design sheet is re-created based on the following assumptions;

1) Storm sewers upstream of the site range from 375 mm to 450 mm in size. Presently a 3 m storm easement conveys the 100-year storm flows from a 0.11 ha residential development and conveys it through an inlet headwall (EX. HW2), to the existing storm sewer network and ultimately discharges into Lake Ontario.
2) Pipe size, lengths and slope are based on topographical surveyed data. Inverts at EX. HW1 and EX. HW2 were retrieved from the Plan and Profile drawings received from the Region of Peel.
3) The existing storm sewer only conveys minor flows (except at the inlet of EX. HW2). From the topographical survey and plan and profile it was noted that all CB's located on Port St. East and Helene St. South are not connected to the existing STM network.

3.2. Proposed Storm Servicing

The storm drainage concept for the site has been designed to maintain flows and contributing drainage areas to the existing outlets on the site as described in Section 3.1. The release rate to

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the municipal storm system from the existing development is based on the 10-year peak flow rate, applying the existing conditions' runoff coefficient (up to a maximum runoff coefficient of 0.5). This was found to be 32 L/s. Under proposed conditions flows from the subject lands will be captured at low points within the site and conveyed through the underground parking lot into EX. MH49. The existing structures within the site will be removed.

A weighted run-off coefficient of 0.67 was used to calculate proposed flows.

Table 1: Area breakdown and run-off coefficient

	Drainage Area (m^2)	Run-off Coefficient C
Impervious Rooftop	709	0.9
Green Roof	410	0.25
Landscaped Area	420	0.25
Impervious Surfaces (includes hard landscaping area)	773	0.9
Overall Site	**0.23**	**0.67**

The existing condition and post development flows from the subject site are shown in Table 2.

Table 2: Existing and Proposed Conditions flows

Outlet Point	Drainage Area	Runoff Coefficient	Description	Existing Condition Flows L/s	
	(ha)			Return Period (Years)	
				10	100*
Existing Condition	0.23	0.5	Conveyed to existing STM network via CB's	32.0	56.0
Post Development	0.23	0.67	Drains to low points within the site and outlets into EX. MH 49	42.0	75.0

* Per City of Mississauga guidelines, a 1.25 adjustment factor is incorporated in calculating the 100-year flow

The 10-year and 100-year design storm event flows were calculated using the rainfall intensity equation: $I\ (mm / hr) = A / (T+B)^C$, where T is the Time of Concentration in minutes. The values for the A, B and C parameters for the various storms were obtained from the latest Engineering Design Criteria from the City of Mississauga, with an initial time of concentration set at 15 minutes.

Under existing conditions, the 450mm diameter storm sewer downstream of EX. MH 49 with a slope of 0.25% (Plan and Profile – Region of Peel) has been estimated to be at 50% capacity (determined through Existing Conditions STM sewer design sheet). Under post development conditions, it is proposed that the municipal storm sewers will convey all flows up to the 100-year event. This added flow increases the pipe capacity to 80% (determined through post development STM sewer design sheet). Since this is the last leg of the sewer before it outlets into Lake Ontario there are no anticipated impacts due to the increase in flow.

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The post development storm sewer design sheet is included in **Appendix B**. Drawing **G-4 Existing Storm Drainage Plan** prepared by Urban Ecosystems (February 2001) is included with this report. Servicing details are included in drawing **GSP-1.**

3.3. Storm Water Management

3.3.1. Water balance and LID Measures

In order to meet the design criteria described in the T&W Developments Requirements Manual, the first 5 mm of runoff should be retained on-site. An annual water balance was established to determine the runoff and infiltration volume under post development conditions with mitigation measures.

Figure 1 indicates the approximate relationship between the percent of total annual average depth vs daily rainfall amounts. The information provided in this graph can be applied to the water balance analysis to demonstrate that 5 mm of runoff is retained on site. For instance, if 50% of the total average annual rainfall is retained on site, the primary target of retaining 5 mm of the daily rainfall depth from all surfaces is achieved. The average annual rainfall depth is 786* mm hence the maximum allowable annual runoff volume for the site is 50% X 786 = 393 mm.

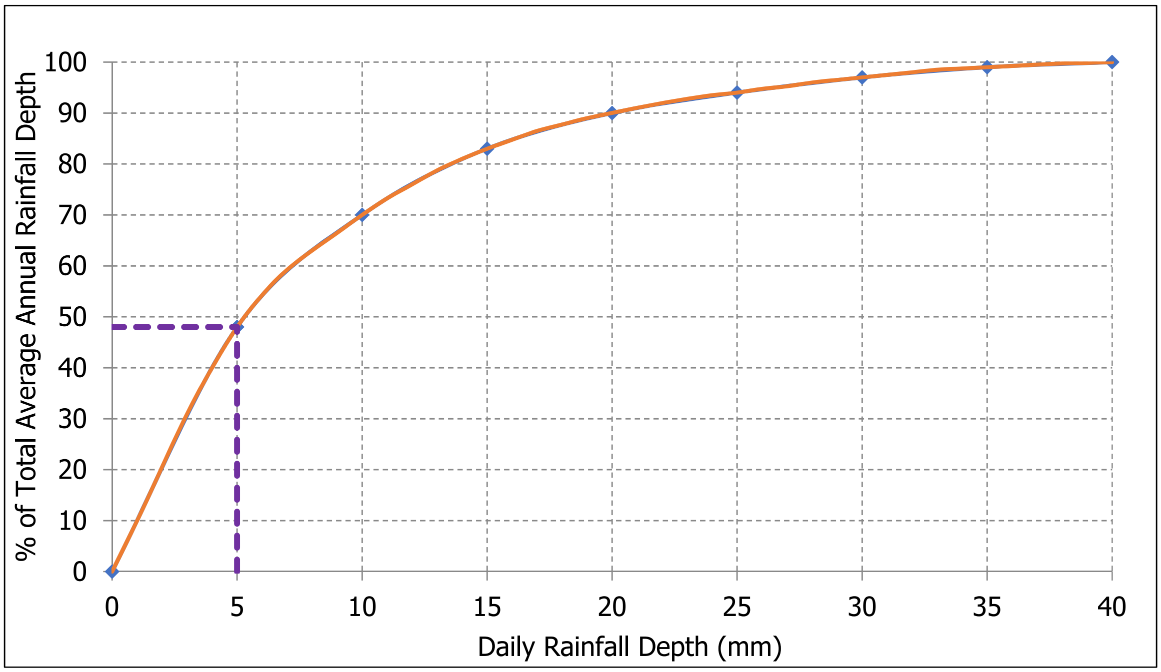

Figure 1: Percent of Total Average Annual Rainfall Depth vs Daily Rainfall Depth (Source: Wet Weather Flow Guidelines, November 2006)

*Rainfall Data Source – Weather station TORONTO LESTER B. PEARSON INT'L A
http://climate.weather.gc.ca/climate_normals/results_1981_2010_e.html?searchType=stnProx&txtRadius=25&optProxType=city&selCity=43%7C35%7C79%7C37%7CMississauga&selPark=&txtCentralLatDeg=&txtCentralLatMin=0&txtCentralLatSec=0&txtCentralLongDeg=&txtCentralLongMin=0&txtCentralLongSec=0&stnID=5097&dispBack=0

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Due to varying land-use types, another approach to achieving this objective is to retain larger storm depths from viable surfaces of the site such that a composite retention of 50% is achieved. The following SWM strategy is proposed to retain 5 mm (equivalent to 50% of annual volume) on site.

- Green Roofs (extensive green rooftop with 15 cm depth or less growing medium)

  Per the Low Impact Development Stormwater Management Planning and Design Guide (2010), "green roofs can help achieve water balance objectives by reducing the total annual runoff volumes. A conservative runoff reduction rate for green roofs is 45-50%" (CVC & TRCA, 2010). Hence green roofs can retain the first 5 mm from every storm event.

- Increased topsoil depth (to be implemented across the site in all landscape surfaces)

  Landscape surfaces can retain rainfall through on-site infiltration and evapotranspiration. To reduce the runoff potential, 150 mm of increased topsoil depth is proposed in all landscape areas under ultimate condition. A topsoil depth of 150 mm over the landscape area with a 40% void space results in a storage volume of approximately 25.2 m^3. Therefore, sufficient storage is available within the void space of the topsoil for the first 15 mm of rainfall from every storm event.

- At a ponding depth of 10 mm (which is lower than the allowable ponding depth of 150), 7 m^3 of storage can be provided on the impervious rooftop surface.

- 1.5 mm of rainfall (equivalent to 15% annual volume) is lost to evaporation from impervious surfaces (includes hard landscaping).

With this design, the maximum runoff retention potential for this site is 50.1 %. Table 2 summarizes the rainfall retention (depth and volume) per surface type for the site. Detailed calculations are attached in Appendix B.

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Table 3 Rainfall Retention (depth and volume) based on Land-Use Type

Land-Use	Catchment Area (m^2)	Description	Storage Volume - Water Balance (m^3) Storage Provided = Area of Catchment [m^2] x depth of rainfall [mm]
Impervious Rooftop	709	First 10 mm of rainfall to be captured and retained on impervious rooftop surfaces	709 m^2 x 10 mm = 7.09 m^3
Green Rooftop	410	First 5 mm of rainfall to be captured and lost through evaporation and infiltration	410 m^2 x 5 mm = 2.05 m^3
Landscape Area	420	First 15 mm of rainfall to be infiltrated within 150 mm depth of additional topsoil.	420 m^2 x 15 mm = 6.30 m^3
Asphalt Parking and Other Impervious Surfaces	773	Uncontrolled runoff with 1.5 mm of rainfall retained on site and lost to evaporation.	773 m^2 x 1.5 mm = 1.16 m^3
	2,312		

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3.3.2. Quantity and Quality Control

Post-development flows from the roof areas, loading bay and other impervious surfaces will be directed to capture points and will be conveyed through the underground parking lot and will outlet into EX. MH49. No Quantity control is required to facilitate the site (CVC SWM Criteria, August 2012). Minor and Major system flows generated from the site are conveyed through the municipal storm sewers and directly outlet into Lake Ontario.

Because the post-development flows mainly consist of "clean" rooftop water and landscape (soft and hard) areas with marginal flows conveyed through the loading bay, no quality control measures are proposed.

4. Sanitary Servicing

4.1. Wastewater Servicing Design Criteria

Wastewater infrastructure will be designed in accordance with the latest Region of Peel Sanitary Sewer Design standards and specifications:

<u>Wastewater Design Criteria</u>

- Type of Development: 1 Bedroom Apartment – 1.68 person/unit
  2+Bedroom Apartment – 2.54 person/unit
- Domestic sewage flow for less than 1000 persons: 0.013 m^3/s

4.2. Existing Wastewater Infrastructure

The existing 450 mm wastewater sewer along Port Street East is the designated gravity outlet for wastewater servicing of the subject lands. Sanitary drainage is captured from the site and conveyed via 250mm diameter sanitary sewer to the existing control manhole outlet at Port Street East, north of the site.

4.3. Proposed Wastewater Servicing

The existing 250mm sanitary sewer has sufficient capacity to convey flows from the proposed development. Water and Wastewater Calculations prepared by MES Engineering has been included in **Appendix C.** The servicing details are included in drawing **GSP-1.**

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5. Water Distribution

Water servicing for the development will conform to the Region of Peel Watermain Design Criteria (2010).

There are three existing fire hydrants in the immediate vicinity of the proposed development; one immediately north of the site on Port Street East, one north east of the subject lands on Port Street East and one North West off the intersection of Port Street East and Helene Street South.

Hydraulic Analysis is required to determine if the fire hydrant located in the immediate vicinity on Port Street can provide adequate fire protection to the site. Calculations prepared by MES Engineering has been included in **Appendix C**.

Refer to Drawing **GSP-1** for further details.

6. Erosion and Sediment Control

The erosion and sediment control plan for the site will be designed in conformance with the City of Mississauga guidelines and Credit Valley Conservation Authority.

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Conclusion

The proposed residential development at Port Street East and Helene Street South can be serviced via the existing storm sewer, sanitary sewer and watermain on Port Street East. The development does not adversely impact any of the surrounding infrastructure or residential development.

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Appendix A:

Geotechnical Investigation (Terraprobe Inc, August 2018)

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Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL
HIGH-RISE BUILDING
55 PORT STREET EAST
CITY OF MISSISSAUGA, ONTARIO

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File No. 1-18-0012-01
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1 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by Brown Maple Investments Ltd. to conduct a geotechnical investigation for a proposed high-rise residential building to be constructed at 55 Port Street East, in the City of Mississauga, Ontario.

This report encompasses the results of the geotechnical investigation conducted for the proposed development site to determine the prevailing subsurface soil and ground water conditions, and on this basis, provides geotechnical engineering design advice and recommendations for the design of building foundations, basement floor slab, earthquake and earth pressure design parameters, basement drainage, shoring and pavement design. In addition, comments are also included on pertinent construction aspects including excavation, backfill and ground water control.

Terraprobe has also conducted a hydrogeological study, Phase 1 and 2 Environmental Site Assessment (ESA) and Record of Site Condition for this property. The findings of the investigations are reported under separate covers.

2 SITE AND PROJECT DESCRIPTION

The subject property is located in the southeast quadrant of the intersection of Port Street East and Helene Street South, in the City of Mississauga, Ontario. The municipal address for the Property is 55 Port Street East, Mississauga, Ontario. The property comprises a roughly rectangular shaped parcel of land, covering approximately 0.23 ha (0.57 acres) area. The property is currently occupied by two and half storey commercial/light industrial building, with associated asphalt paved parking areas, driveways/access routes and landscaping areas. The general location of the site is presented on Figure 1.

It is proposed to demolish the existing building to facilitate the redevelopment of the property to include a ten-storey building with one level of underground parking garage across the project site. The development would be serviced by municipal water and sewers.

3 INVESTIGATION PROCEDURE

The field investigation was conducted during the period of March 21 to 23, 2018 and consisted of drilling and sampling a total of five (5) boreholes to depths ranging from 8.9 to 12.3 m below grade within the footprints of the proposed building and the underground parking garage. The approximate locations of the boreholes are shown on the enclosed Borehole Location Plans (Figures 2A and 2B).

The boreholes were drilled by a specialist drilling contractor using a track-mounted drill rig power auger. The borings were advanced using continuous flight solid stem augers and were sampled at 0.75 m (up to 3.0 m depth) and 1.5 m (below 3.0 m depth) intervals with a conventional 50 mm diameter split barrel sampler when the Standard Penetration Test (SPT) was carried out (ASTM D1586). The field work (drilling, sampling and testing) was observed and recorded by a member of our field engineering staff, who logged the borings and examined the samples as they were obtained.

All samples obtained during the investigation were sealed into clean plastic jars, and transported to our geotechnical testing laboratory for detailed inspection and testing. All borehole samples were examined (tactile) in detail by a geotechnical engineer, and classified according to visual and index properties. Laboratory tests consisted of water content determination on all samples; and a Sieve and Hydrometer analysis on eight (8) selected native soil samples (Borehole 1, Samples 4B and 6; Borehole 2, Samples 2 and 7; Borehole 3, Samples 3 and 6; Borehole 4, Sample 4; and Borehole 5, Sample 7) and Atterberg Limits test on four (4) selected samples (Borehole 1, Sample 6; Borehole 2, Sample 7; Borehole 3, Sample 6; and Borehole 5, Sample 7). The measured natural water contents of individual samples and the results of the Sieve and Hydrometer analysis and Atterberg Limits tests are plotted on the enclosed Borehole Logs at respective sampling depths. The results of Sieve and Hydrometer analysis and Atterberg Limits tests are also summarized in Section 4.7 of this report, and appended.

Water levels were measured in open boreholes upon completion of drilling. Monitoring wells comprising 50 mm diameter PVC pipes were installed in Boreholes 1, 2, 4 and 5 to facilitate ground water monitoring and for the purpose of hydrogeological study. The PVC tubing was fitted with a bentonite clay seal as shown on the accompanying Borehole Logs. Water levels in the monitoring wells were measured on April 2, 2018 (about one week following completion of the installation). The results of ground water monitoring are presented in Section 4.8 of this report.

The borehole ground surface elevations were surveyed by Terraprobe using a Trimble R10 GNSS System. The Trimble R10 system uses the Global Navigation Satellite System and the Can-Net reference system to determine target location and elevation. The Trimble R10 system is reported to have an accuracy of up to 10 mm horizontally and up to 30 mm vertically.

It is should be noted that the elevations provided on the Borehole Log are approximate, for the purpose of relating soil stratigraphy and should not be used or relied on for other purposes.

4 SUBSURFACE CONDITIONS

The specific soil conditions encountered at each borehole location are described in greater detail on the Borehole Logs, with a summary of the general subsurface soil conditions outlined below. This summary is intended to correlate this data to assist in the interpretation of the subsurface conditions encountered at the site.

It should be noted that the subsurface conditions are confirmed at the borehole locations only, and may vary between and beyond the borehole locations. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of geologic change.

4.1 Surficial Layers

An asphalt pavement structure was encountered in Boreholes 1 to 3 and consisted of 80 to 90 mm thick asphaltic concrete underlain by 120 to 670 mm thick granular base/subbase course.

A topsoil layer was encountered at the ground surface in Boreholes 4 and 5. The topsoil thickness was about 100 mm.

The above topsoil and asphalt pavement thicknesses were measured from the borehole drilling and are approximate. We recommend that a shallow test pit investigation be carried out to determine precise topsoil and pavement thickness present across the site for quantity estimation and costing purposes (if required).

4.2 Earth Fill

Earth fill materials, consisting of clayey silt to sand/silty sand were encountered beneath the topsoil layer or pavement structure in Boreholes 3 to 5 and extended to about 0.8 m (Borehole 3) to 2.3 m (Boreholes 4 and 5) depth below grade. The earth fill materials generally consist of trace amounts of organic matters.

Standard Penetration Test results (N-values) obtained from the clayey silt earth fill zone ranged from 10 to 12 blows per 300 mm of penetration, indicating a stiff consistency. The in-situ moisture contents of the earth fill samples ranged from 10 to 13 percent by mass, indicating a moist condition.

N-values obtained from the sand/silty sand earth fill zone ranged from 1 to 14 blows per 300 mm of penetration, indicating a very loose to compact relative density. The in-situ moisture contents of the earth fill samples ranged from 5 to 20 percent by mass, indicating a moist to locally wet condition.

4.3 Sandy Silt to Silt and Sand/Sand/Sandy Gravel

Cohesionless soil deposits, consisting of sandy silt to silt and sand/sand/sandy gravel encountered beneath the pavement structure or the earth fill zone in each borehole and extended to about 2.3 m (Boreholes 2 and 3) to 4.6 m (Borehole 5) depth below grade.

N-values obtained from the cohesionless soils ranged from 7 to 41 blows per 300 mm of penetration, indicating a loose to dense relative density. The in-situ moisture contents of the samples ranged from 8 to 22 percent by mass, indicating a moist to wet condition.

4.4 Silt

Silt deposit, with some clay and tract to some sand was encountered beneath the cohesionless soil deposits in Boreholes 1 to 4 and extended to about 3.0 m (Borehole 3) to 4.6 m (Boreholes 1 and 2) depth below grade.

N-values obtained from the silt deposit ranged from 7 to 23 blows per 300 mm of penetration, indicating a loose to compact relative density. The in-situ moisture contents of the samples ranged from 14 to 17 percent by mass, indicating a moist to wet condition.

4.5 Glacial Till

Clayey silt till deposit, with varying amounts of sand (some sand to sandy) and trace to some gravel was encountered beneath the silt deposit at depths of 3.0 to 4.6 m below grade in Boreholes 1 to 4 and the sandy gravel deposit at 4.6 m depth below grade in Borehole 5 and extended to 8.8 m (Borehole 4) to 10.7 m (Boreholes 1 to 3 and 5) depth below grade.

N-values obtained from the till deposit ranged from 8 to 83 blows per 300 mm of penetration to 50 blows per 125 mm of penetration, indicating a very stiff to hard consistency. The in-situ moisture contents of the glacial till samples ranged from 8 to 23 percent by mass, indicating a moist condition.

It should be noted that the glacial till deposit may contain larger size particles (cobbles and boulders) that are not specifically identified in the boreholes. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples for the particles of this size.

4.6 Inferred Bedrock

The glacial till deposits graded into inferred weathered shale (Bedrock of Georgian Bay Formation) at 8.8 m (Borehole 4) to 10.7 m (Boreholes 1 to 3 and 5) depth below grade and extended about 1m or less into inferred shale. The hard resistance could also be due to cobbles or boulders.

The inferred bedrock beneath the site is expected to be of the Georgian Bay Formation, which is a deposit, predominantly comprising thin to medium bedded grey shale of Ordovician age. The shale typically contains interbedded grey calcareous shale, limestone/dolostone and calcareous sandstone (conventionally grouped together as “limestone”) which can be discontinuous and nominally 25 to 125 mm thick.

There is typically a zone of weathering at the contact between the weak rock of the Georgian Bay Formation and the overburden. In the Ontario Ministry of Transportation and Communications document RR229 - *Evaluation of Shales for Construction Projects- an Ontario Shale Rating System*, March 1983, there is reproduced from Skempton, Davis and Chandler, a typical weathering profile of a low durability shale, that characterizes the shale surface into three grades of weathering and four zones described as follows:

Weathered Class	Zone	Description	Notes
Fully Weathered	IV[b]	soil like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IV[a]	soil like matrix with occasional pellets of shale less than 3 mm dia.	little or no trace of rock structure, although matrix may contain relic fissures
	III	soil like matrix with frequent angular shale particles up to 25 mm dia.	moisture content of matrix greater than the shale particles
	II	angular blocks of unweathered shale with virtually no matrix separated by weaker chemically weathered but intact shale	spheroidal chemical weathering of shale pieces emanating from relic joints and fissures, and bedding planes
Unweathered (Sound)	I	shale	regular fissuring

In the Greater Toronto Area (the surface of the rock having been scoured and involved by the base of glacial ice), Shale Zone IV is typically not present in an identifiable form. At the base of the overburden there is usually found a zone of ground rock with a clayey consistency and fragmented shale that corresponds to Zone III in the shale profile, but this zone also typically contains imported drift material. This zone of material can be interpreted as the lowest portion of the till (if present as overburden) or as partially weathered rock of Zone III. This zone of rock with a clayey consistency and fragmented shale appears to vary in thickness within the Greater Toronto area (typically on the order of 1 m). The distinction is subjective and depends on the investigator.

The augered borehole method used at this site is conventionally accepted investigative practice. However, the interval sampling method does not define the bedrock surface with precision, particularly where the surface of the rock is weathered, weaker and easily penetrated by auger. The change in resistance to augering in between Zones II and III in the shale profile is not profound. The auger refusal is generally indicative of a presence of a relatively less weathered/sound shale and/or limestone/dolostone layers. It should be noted that confirmation and characterization of the bedrock through rock coring was not included in our scope of work. Therefore, the bedrock surface elevations at the borehole locations, as noted on the borehole logs, could not be confirmed, and were inferred from the borehole augering, auger grinding, split barrel sampler refusal and bouncing. Auger grinding or sampler refusal in this case could either be inferred as bedrock or could be due to the presence of boulders/obstruction/limestone slabs which may be present within the overburden, therefore actual bedrock surface elevations may vary from the inferred elevations noted on the borehole logs and provided here. It must be noted that inference of bedrock level based on auger grinding and/or sampler refusal does not provide bedrock level accurately.

Any variation in the design bedrock level and actual bedrock level may result in significant cost implications and schedule delays (including redesign and additional construction costs) for the project. We recommend that bedrock level at this site should be established by rock coring to help minimize such risks if the design requires.

4.7 Geotechnical Laboratory Test Results

The geotechnical laboratory testing consisted of natural water content determination for all samples, while a Sieve and Hydrometer analysis and Atterberg Limits tests were conducted on selected soil samples. The test results are plotted on the enclosed Borehole Logs at respective sampling depths.

The results (graphs) of the Sieve and Hydrometer (grain size) analysis are appended and a summary of these results is presented as follows:

Borehole No. Sample No.	Sampling Depth below Grade (m)	Percentage (by mass)				Descriptions (MIT System)
		Gravel	Sand	Silt	Clay	
Borehole 1, Sample 4B	2.7	0	7	75	18	SILT some clay, trace sand
Borehole 1, Sample 6	4.9	12	28	39	21	CLAYEY SILT TILL sandy, some gravel
Borehole 2, Sample 2	1.1	0	43	53	4	SILT AND SAND trace clay
Borehole 2, Sample 7	6.4	10	28	40	22	CLAYEY SILT TILL sandy, trace gravel
Borehole 3, Sample 3	1.8	0	46	52	2	SILT AND SAND trace clay
Borehole 3, Sample 6	4.9	17	28	37	18	SANDY SILT TILL some clay, some gravel
Borehole 4, Sample 4	2.6	0	99	1		SAND trace silt
Borehole 5, Sample 7	6.4	10	26	41	23	CLAYEY SILT TILL sandy, trace gravel

Atterberg Limits Tests were also carried out on the above selected soil samples. The results were plotted on A-Line Graph (refer to enclosed Figure, Atterberg Limits Test Results) and summarized as follows:

Borehole No. Sample No.	Sampling Depth below Grade (m)	Liquid Limit (W_L)	Plastic Limit (W_P)	Plasticity Index (I_P)	Natural Water Content (percent)	Plasticity
Borehole 1, Sample 6	4.9	24	15	9	12	Slightly Plastic
Borehole 2, Sample 7	6.4	24	15	9	na	Slightly Plastic
Borehole 3, Sample 6	4.9	24	16	8	12	Slightly Plastic
Borehole 5, Sample 7	6.4	24	14	10	8	Slightly Plastic

4.8 Ground Water

Observations pertaining to the depth of water level and caving were made in the open boreholes immediately after completion of drilling, and are noted on the enclosed Borehole Logs. Monitoring wells were installed in Boreholes 1, 2, 4 and 5 to facilitate ground water level monitoring and for the purpose of the hydrogeological study. The ground water level measurements in the monitoring wells were taken on April 2, 2018 (about one week following completion of the installation) and are noted on the enclosed Borehole Logs. A summary of these observations is provided as follows:

Borehole No.	Depth of Boring below Grade	Depth to Cave below Grade	Water Level Depth/Elevation at the Time of Drilling	Water Level Depth/Elevation in Monitoring Well on April 2, 2018
Borehole 1	12.3 m	Open	10.4 m/67.8 m	6.5 m/71.7 m
Borehole 2	10.7 m	Open	10.5 m/67.7 m	6.8 m /71.5 m
Borehole 3	10.8 m	Open	10.4 m/67.8 m	Monitoring well not installed
Borehole 4	8.9 m	Open	3.0 m/74.7 m	3.2 m/74.6 m
Borehole 5	10.7 m	4.6 m	3.0 m/74.6 m	3.0 m/74.7 m

The water levels noted above may fluctuate seasonally depending upon the precipitation and surface runoff. The water levels may be about 600 mm higher than the water levels noted above, where capillary rise may occur in the cohesionless silt/sand soils.

It is likely that the long-term stabilized ground water level is close to the water level in nearby Lake Ontario (approx. Elev. 75 m Geodetic).

5 DISCUSSIONS AND RECOMMENDATIONS

The following discussion and recommendations are based on the factual data obtained from this investigation and are intended for the use of the owner and the design engineer. Contractors bidding or providing services on this project should review the factual data and determine their own conclusions regarding construction methods and scheduling.

This report is provided on the basis of these terms of reference and on the assumption that the design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features or there is any additional information relevant to the interpretations made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Terraprobe should be retained to review the implications of these changes with respect to the contents of this report.

5.1 Foundation

All boreholes are located within the footprints of the high-rise building and the underground parking garage. These boreholes encountered surficial layers at the ground surface and the earth fill zone extending to 0.8 to 2.3 m depth below grade, underlain by cohesionless soil and silt deposits extending to 3.0 to 4.6 m depth below, which is in turn underlain by clayey silt till deposit extending to about 8.8 to 10.7 m depth below grade. The glacial till deposit graded into inferred shale bedrock, or cobbles, or boulders which were penetrated by about 1m or less in the boreholes.

It is understood that the proposed building would be a ten-storey building with one level of underground parking garage. Based on the design drawing provided by Giannone Petricone Associates Inc. (55 Port Street East, Proposed Summary: Section & Key Plan, 12, dated September 29, 2017), the finished basement floor would be set at 3,735 mm below grade. Therefore, the average finished basement floor elevation would be at Elev. 74.0 m ± (average ground elevation of the project site, Elev. 78.0 m ±) and the underside of building foundation would likely be designed at about Elev. 73.0 m. The following table summarizes the recommended geotechnical reaction and geotechnical resistance available at the borehole locations.

BH No.	Highest (Bottom) of Footing Elevation	Max. Geotechnical Reaction at SLS (kPa)	Max. Factored Geotechnical Resistance at ULS (kPa)	Water Level Measured in Monitoring Wells (m)	Bearing Stratum
1	73.0 m 72.0 m	300 500	450 750	6.5 m/71.7 m	Very Stiff Clayey Silt Till Hard Clayey Silt Till
2	73.0 m 72.0 m	300 500	450 750	6.8 m /71.5 m	Stiff Clayey Silt Till Hard Clayey Silt Till
3	72.6 m 72.0 m	300 500	450 750	na	Stiff Sandy Silt Till (some clay) Hard Clayey Silt Till

BH No.	Highest (Bottom) of Footing Elevation	Max. Geotechnical Reaction at SLS (kPa)	Max. Factored Geotechnical Resistance at ULS (kPa)	Water Level Measured in Monitoring Wells (m)	Bearing Stratum
4	73.0 m 72.0 m	300 500	450 750	3.2 m/74.6 m	Very Stiff Clayey Silt Till Very Stiff to Hard Clayey Silt Till
5	71.5 m	300	450	3.0 m/74.7 m	Hard Clayey Silt Till

Notes: ULS=Ultimate Limit States; and SLS=Serviceability Limit States

The above design bearing pressures as recommended allow for up to 25 mm of total settlement. This settlement will occur as load is applied and is linear elastic and non-recoverable. Differential settlement is a function of spacing, loading and foundation size.

If the foundation for this building has to be extended to relatively deeper depth to be supported on more competent clayey silt till deposit with relatively higher bearing capacity, the over-excavation required for the foundation in this area may be filled with lean mix concrete (strength to be provided by the structural engineer) up to the normal design foundation level, and the foundation may be supported on this lean mix concrete pad. The lean mix concrete pad must extend a minimum of 300 mm beyond the edge of the foundation in every direction. It is recommended that the final foundation design drawings be reviewed by a geotechnical engineer to ensure that geotechnical recommendations including the soil bearing capacities provided above have been conformed to, as required.

The east limit of the proposed building basement is adjacent to the west limit of the existing building basement located at 65 Port Street East. Based on the design drawing provided by Giannone Petricone Associates Inc. (55 Port Street East, Floor Plan: Parking Plan, 04, dated September 29, 2017), it is understood that the finished basement floor elevation for the proposed building would be set at similar elevation to that of the adjacent existing basement floor. However, if foundation excavation is to be extended deeper than the existing basement foundation level, appropriate excavation shoring must be provided to maintain the integrity of the existing building foundations. The detailed shoring recommendations are provided in Section 5.9.

5.1.1 Foundation Installation

All exterior foundations and foundations in unheated areas must be provided with a minimum soil cover of 1.2 m or equivalent insulation for frost protection.

It is recommended that all excavated footing base must be evaluated by a qualified geotechnical engineer to ensure that the founding soils exposed at the excavation base are consistent with the design bearing pressure intended by the geotechnical engineer.

Prior to pouring foundation concrete, the foundation subgrade should be cleaned of all deleterious materials such as topsoil, fill, softened, disturbed or caved materials, as well as any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the foundation subgrade and concrete must be provided.

It is noted that the native soils tend to weather rapidly and deteriorate on exposure to the atmosphere or surface water. Hence, foundation bases which remain open for an extended period of time should be protected by a skim coat of lean concrete. Provisions should be made to minimize disturbance to the exposed foundation subgrade.

5.2 Basement Floor Slab

The excavated surface should be assessed by a qualified geotechnical engineer. The modulus of subgrade reaction appropriate for the slab design constructed is provided as follows,

K_s =30,000 kPa/m (undisturbed clayey silt till deposits)
K_s =30,000 kPa/m (undisturbed silt deposits)
K_s =50,000 kPa/m (sandy gravel)

The basement floor slab should be provided with a capillary moisture barrier and drainage layer. This can be made by placing the slab on a minimum 200 mm thick 19 mm clear stone layer (OPSS MUNI 1004) compacted by vibration to a dense state. This material also serves as the drainage media for the subfloor drainage system. Provision of subfloor drainage is required in conjunction with the perimeter drainage of the structure.

The subfloor drainage system is an important building element, as such the storm sumps which ensure the performance of this system must have a duplexed pump arrangement for 100 percent pumping redundancy provided with emergency power. Basement and subfloor drainage provisions are further discussed in Section 5.7 of this report.

5.3 Excavations and Ground Water Control

The boreholes data indicate that the earth fill/weathered/disturbed materials and undisturbed native soils would be encountered in the excavations. Excavations must be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety.

TYPE 1 SOIL

a. is hard, very dense and only able to be penetrated with difficulty by a small sharp object;
b. has a low natural moisture content and a high degree of internal strength;
c. has no signs of water seepage; and
d. can be excavated only by mechanical equipment.

TYPE 2 SOIL

a. is very stiff, dense and can be penetrated with moderate difficulty by a small sharp object;
b. has a low to medium natural moisture content and a medium degree of internal strength; and
c. has a damp appearance after it is excavated.

TYPE 3 SOIL

a. is stiff to firm and compact to loose in consistency or is previously-excavated soil;
b. exhibits signs of surface cracking;
c. exhibits signs of water seepage;
d. if it is dry, may run easily into a well-defined conical pile; and
e. has a low degree of internal strength

TYPE 4 SOIL

a. is soft to very soft and very loose in consistency, very sensitive and upon disturbance is significantly reduced in natural strength;
b. runs easily or flows, unless it is completely supported before excavating procedures;
c. has almost no internal strength;
d. is wet or muddy; and
e. exerts substantial fluid pressure on its supporting system.

The earth fill materials, silt and cohensionless deposits (consisting of sandy silt to silty sand, sand and sandy gravel) encountered in the boreholes are classified as Type 3 Soil, while the undisturbed native clayey silt till deposit would be generally classified as Type 2 Soil above and Type 3 Soil below prevailing ground water level, under these regulations.

Where workmen must enter excavations advanced deeper than 1.2 m, the trench walls should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. The regulation stipulates the steepest slopes of excavation by soil type as follows:

Soil Type	Base of Slope	Steepest Slope Inclination
1	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
2	within 1.2 metres of bottom of trench	1 horizontal to 1 vertical
3	from bottom of trench	1 horizontal to 1 vertical
4	from bottom of trench	3 horizontal to 1 vertical

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act and Regulations for Construction Projects, and include provisions for timbering, shoring and moveable trench boxes.

It should be noted that the glacial till deposit may contain larger particles (cobbles and boulders) that are not specifically identified in the Borehole Logs. The size and distribution of such obstructions cannot be predicted with borings, because the borehole sampler size is insufficient to secure representative samples of the particles of this size. Provision should be made in excavation contracts to allocate risks associated with time spent and equipment utilized to remove or penetrate such obstructions when encountered.

Although it is not expected that excavations extend into the inferred bedrock, the George Bay Formation is a rippable rock that can be removed with conventional excavation equipment once it has been displaced by a ripper tooth or hoe ram.

Terraprobe had previously completed two geotechnical investigations in a close proximity to this property in 1992 and 2000 and advanced a total of 15 boreholes to depths of about 9 to 15 m below grade. The previous borehole data indicate that the general area may be partially underlain by possible undisturbed organic soils. Although organic soil was not encountered in our recent site-specific boreholes, if the organic soil is encountered during the foundation excavation, a qualified geotechnical engineer should be retained to evaluate the soil conditions and provide recommendations for building foundations and geotechnical design and further tests may be required.

Terraprobe has completed the Hydrogeological Report (File No. 1-18-0012-46) for this site to provide ground water control measures and estimate ground water discharge volume (Refer to this report for detailed information).

The ground water levels measured in the monitoring wells (Boreholes 1, 2, 4 and 5) on April 2, 2018 indicated that the water levels generally ranged from about Elev. 71.5 m to Elev. 74.7 m. Due to the close proximity of Lake Ontario, it can be expected that long-term ground water levels would reach near Elev. 75.0 m Geodetic.

Perched ground water seepage may be encountered during the excavations primarily emanating from the fill materials, sand/sandy gravel and silt deposits. The perched ground water seepage should diminish slowly and can be controlled by continuous pumping from a conventional sump and pump arrangement at the base of the excavation. For excavations extending to depths greater than 0.3 m below the prevailing water table, it will be necessary to lower the ground water level below the excavation base, prior to, and maintain during the subsurface construction.

5.4 Backfill

The native soils are considered suitable for backfill provided the moisture content of these soils is within 3 percent of the Optimum Moisture Content (OMC). It should be noted that there may be wet zones within the subsurface soils which could be too wet to compact. Any soil material with 3 percent or higher in-situ moisture content than its OMC, could be put aside to dry or be tilled to reduce the moisture content so that it can be effectively compacted. Alternatively, materials of higher moisture content could be wasted and replaced with imported material which can be readily compacted.

In settlement sensitive areas, the backfill should consist of clean earth and should be placed in lifts of 150 mm thickness or less, and heavily compacted to a minimum of 95 percent Standard Proctor

Maximum Dry Density (SPMDD) at a water content close to OMC (within 3 percent). The upper 1.2 m of the pavement subgrade must be compacted to a minimum of 98 percent SPMDD.

It should be noted that the soils encountered on the site are generally not free draining, and will be difficult to handle and compact should they become wetter as a result of inclement weather or seepage. Hence, it can be expected that the earthworks will be difficult and may incur additional costs if carried out during wet periods (i.e. spring and fall) of the year.

5.5 Earth Pressure Design Parameters

Walls or bracings subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$\mathbf{P = K\,[\gamma\,(h-h_w) + \gamma' h_w + q] + \gamma_w h_w}$$

Where:

- $\mathbf{P}$ = the horizontal pressure (kPa)
- $\mathbf{K}$ = the earth pressure coefficient
- $\mathbf{h}$ = the depth below the ground surface (m)
- $\mathbf{h_w}$ = the depth below the ground water level (m)
- $\boldsymbol{\gamma}$ = the bulk unit weight of soil (kN/m^3)
- $\boldsymbol{\gamma_w}$ = the bulk unit weight of water (9.8 kN/m^3)
- $\boldsymbol{\gamma'}$ = the submerged unit weight of the exterior soil, (γ_{sat} - γ_w)
- $\mathbf{q}$ = the complete surcharge loading (kPa)

Where the wall backfill can be drained effectively to eliminate hydrostatic pressures on the wall, this equation can be simplified to:

$$\mathbf{P = K[\gamma h + q]}$$

This equation assumes that free-draining granular backfill is used and positive drainage is provided to ensure that there is no hydrostatic pressure acting in conjunction with the earth pressure.

Earth pressure distribution information for one-level support system such as excavation shoring design is provided in Section 5.9 of this report.

Resistance to sliding of retaining structures is developed by friction between the base of the footing and the soil. This friction **(R)** depends on the normal load on the soil contact (**N**) and the frictional resistance of the soil (**tan ϕ**) expressed as **R = N tan ϕ**. The factored geotechnical resistance at ULS is **0.8 R**.

Passive earth pressure resistance is generally not considered as a resisting force against sliding for conventional retaining structure design because a structure must deflect significantly to develop the full passive resistance.

The average values for use in the design of walls subjected to unbalanced earth pressures at this site are tabulated as follow:

Parameter	Definition	Units
ϕ	angle of internal friction	degrees
γ	bulk unit weight of soil	kN/m^3
K_a	active earth pressure coefficient (Rankine)	dimensionless
K_o	at-rest earth pressure coefficient (Rankine)	dimensionless
K_p	passive earth pressure coefficient (Rankine)	dimensionless

Stratum/Parameter	γ	Φ	K_a	K_o	K_p
Earth Fill	18.0	28	0.36	0.53	2.77
Sandy Silt to Silty Sand	21.0	34	0.28	0.44	3.54
Silt	21.0	30	0.33	0.50	3.00
Sand	22.0	30	0.33	0.50	3.00
Sandy Gravel	22.0	35	0.27	0.43	3.69
Clayey Silt Glacial Till	21.0	32	0.31	0.47	3.25

The above values of the earth pressure coefficients are for the horizontal backfill grade behind the wall. The earth pressure coefficients for inclined grade will vary based on the inclination of the retained ground surface.

5.6 Earthquake Design Parameters

The Ontario Building Code (2012) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A. of the Ontario Building Code (2012). The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity (v_s)

measurements have been taken. Alternatively, the classification is estimated on the basis of rational analysis of undrained shear strength (s_u) or penetration resistance (N-values).

$$\upsilon_{s-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{\upsilon_{si}}} \qquad S_{u-avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{s_{ui}}} \qquad N_{avg} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

Shear Wave Velocity **Undrained Shear Strength** **SPT N-values**

Based on the borehole data (advanced to a maximum depth of about 12.3 m below grade), it is understood that the proposed building will be founded on stiff to hard glacial till deposit, which graded into the inferred shale bedrock at depths of 8.8 to 10.7 m below grade. It is expected that the deeper stratigraphy in this area is as competent as the lowest proven strata in the boreholes. On this basis, site seismic classification may be taken as Site Class C according to Table 4.1.8.4.A of the Ontario Building Code (2012). Tables 4.1.8.4.B. and 4.1.8.4.C. of the Ontario Building Code (2012) provide the applicable acceleration and velocity based site coefficients. The applicable acceleration and velocity based site coefficients for Site Class C are provided as follows:

Site Class	Values of F_a (acceleration based coefficients)				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
C	1.0	1.0	1.0	1.0	1.0

Site Class	Values of F_v (velocity based coefficients)				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
C	1.0	1.0	1.0	1.0	1.0

It should be noted that the above site seismic designation is estimated on the basis of rational analysis of the undrain shear strength obtained from the boreholes advanced at the site to a maximum depth of about 12.3 m below grade. A site specific Multichannel Analysis of Surface Waves (MASW) may be considered to confirm the site seismic classification, if required.

5.7 Basement Drainage

The ground water levels measured in the monitoring wells (Boreholes 1, 2, 4 and 5) on April 2, 2018 generally ranged from about Elev. 71.5 m to Elev. 74.7 m. The average finished basement floor of would be at about Elev. 74.0 m. The nearby Lake Ontario water level is at about Elev. 75.0 m Geodetic.

The exterior grade around the buildings should be sloped away at a 2 percent gradient or more for a distance of at least 1.2 m to assist in maintaining basement dry from seepage. Where the structure is made directly against a shored excavation, drainage is provided by forming a drained cavity with prefabricated drain material, such as CCW MiraDRAIN 6000 series (or Terrafix Terradrain 200, or approved equivalent) which can be incorporated between the shoring and the cast-in-place concrete foundation wall. The drainage composite material can be outlet into the basement sumps using a solid pipe (separate from the subfloor drainage system) to remove collected water at the building sumps. (Refer to enclosed Figure 3 Schematic Basement Drainage)

The sub-floor drainage system should consist of perforated pipes (minimum 100 mm diameter) located at a spacing of about 5.0 m centre to centre (Refer to Figure 4 Basement Floor Subdrain Detail). The subdrain system should be outlet to a suitable discharge point under gravity flow, or connected to a sump located in the basement. The water from the sump must be pumped out to a suitable discharge point/positive outlet. The installation of the drains as well as the outlet must conform to the applicable plumbing code requirements.

The elevator pit would likely extend 1 to 2 m deeper than the lowest basement floor level. Drainage for the elevator pit may be provided by incorporating perimeter and subfloor drainage system outletting to a sump, or the elevator pit structure can be waterproofed below the lowest basement subfloor drainage system level.

The size of the sump should be adequate to accommodate the anticipated water seepage. An industrial duplex pumping arrangement (main pump with a provision of a backup pump) on emergency backup power is recommended. The pump capacity must be adequate to accommodate peak flow conditions expected during the wet seasons (i.e., spring melt and fall). Refer to the Hydrogeological report for ground water seepage rates and volumes.

The subfloor drainage system is an important building element at this site, as such the storm sump that ensures the performance of this system must have an industrial duplexed pump arrangement on emergency power, as noted above, for 100 percent pumping redundancy.

5.8 Pavement

It is understood that at-grade pavements will be constructed on underground parking garage concrete deck. For pavement structure supported on concrete deck, recommendations will be provided during the detailed design stage in consultation with the design team. Design recommendations for the entrance driveway pavement structure (to be supported on soil subgrade) are provided in this section.

5.8.1 Pavement Design

The asphalt pavement design for the entrance driveways supported on soil subgrade is provided in the following table.

Pavement Structural Layers	Driveway
HMA Surface Course, OPSS 1150 HL 3	40 mm
HMA Binder Course, OPSS 1150 HL 8	85 mm
Granular Base Course, OPSS MUNI 1010 Granular A	150 mm
Granular Subbase Course, OPSS MUNI 1010 Granular B Type I	350 mm
Total Thickness	625 mm

5.8.2 Drainage

Control of water is an important factor in achieving a good pavement life. Therefore, we recommend that provisions be made to drain the new pavement subgrade and its granular layers. Continuous pavement subdrains (designed to drain into catchbasins) should also be provided along both sides of the driveway curblines. All sub-drain arrangements should comply with the City of Mississauga Standard Drawing No. 2220.040.

5.8.3 General Pavement Recommendations

It should be noted that in addition to the adherence to the above pavement design recommendations, a close control on the pavement construction process would also be required in order to obtain the desired pavement life. It is recommended that regular inspection and testing be conducted during the pavement construction to confirm material quality, thickness, and to ensure adequate compaction.

HL 3 and HL 8 hot mix asphalt mixes should be designed, produced and placed in conformance with OPSS 1150 and OPSS 310 requirements and the relevant City's requirements.

Both the Granular A and Granular B Type I materials should meet the requirements of OPSS MUNI 1010 requirements and the relevant City's standards. Granular materials should be compacted to 100 percent of SPMDD.

HL3 HS hot mix asphalt is recommended as padding. Padding should be placed in lifts not exceeding 50 mm.

Performance graded asphalt cement, PG 58-28, conforming to OPSS MUNI 1101 requirements, should be used in both HMA binder and surface courses.

A tack coat (SS1) should be applied to all construction joints prior to placing hot mix asphalt to create an adhesive bond. SS1 tack coat should also be applied between hot mix asphalt binder and surface courses.

5.8.4 Subgrade Preparation

The exposed subgrade is expected to generally consist of native clayey silt or fill materials and these soils will be weakened by construction traffic when wet; especially if site work is carried out during periods of wet weather. In these weather conditions, an adequate granular working surface would be required in order to minimize subgrade disturbance and protect its integrity.

Immediately prior to placing the granular subbase, the exposed subgrade should be proof rolled with a heavy rubber tired vehicle (such as a loaded gravel truck). The subgrade should be inspected for signs of rutting or displacement. Areas displaying signs of rutting or displacement should be compacted and tested or the material should be excavated and replaced with the Granular B Type I. Backfill material should be placed and compacted to at least 100 percent of SPMDD. The final subgrade surface should be sloped at a grade of 3 percent to provide positive subgrade drainage.

5.9 Shoring Design Consideration

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary shoring system design should be carried out by a licensed Professional Engineer experienced in shoring design.

It is understood that the finished basement floor elevation of the proposed building would be at the similar level to that of the existing basement floor elevation at 65 Port Street East. The east limit of the proposed building basement is adjacent to the west limit of the existing building basement. Therefore, special attention should be made along the proposed excavation shoring sections adjacent to the limits of the existing basement. No excavation shall extend below the foundations of the existing adjacent building without adequate alternative support being provided. See Figure 5 for guidelines related to underpinning of existing foundations.

The shoring requirements for the site will have to be examined in detail with respect to the proximity of existing structures and site boundary constraints. Depending upon the site conditions, the shoring system may need to consist of a rigid (interlocking drilled caissons) or a steel soldier piles and timber lagging shoring system, or a combination of both. The site conditions must be carefully assessed by the shoring designer to select appropriate type of shoring system in light of the close proximity of the existing high-rise buildings. It is imperative that the shoring system provides adequate support to the existing building foundations.

5.9.1 Earth Pressure Distribution

Applicable soil parameters are included in the Earth Pressure Design Parameters Section (Section 5.5).

Where a single level of support may be required for shoring system for one level of basement, a triangular earth pressure distribution similar to that used for the basement wall design is appropriate for this case.

$$P = K(\gamma h + q)$$

Where:

- **P =** the horizontal pressure (kPa)
- **K =** the earth pressure coefficient
- **h =** excavation depth below surface (m)
- **γ =** the bulk unit weight of the soil (kN/m^3)
- **q =** the complete surcharge loading (kPa)

5.9.2 Soldier Pile Toe Design

It is envisaged that the soldier pile will be generally socketed in the clayey silt till or shale bedrock. The horizontal resistance of the soldier pile toes will be developed by the embedment below the base of excavation where resistance is developed from passive earth pressure. It is noted that where soils exist beneath the ground water level, the unit weight of the soil is diminished by buoyancy, and therefore, the resistance from these soils will be different depending on whether the soils are dewatered, or remain below the nominal ground water level. The design of the shoring should therefore consider the construction plan and sequence with respect to depth of ground water control. There may be zones of material within the subsurface soils which may be wet and permeable such that augered borings for soldier piles made into these soils may be unstable. In these cases, it will be necessary to advance temporarily cased holes to prevent excess caving during the soldier pile installations.

5.9.3 Shoring Support

It will be necessary to secure encroachment agreements from the City and the adjacent land owners, in order to use soil anchors on the adjacent properties. Pre-construction condition surveys should be carried out for the adjacent structures to establish existing conditions prior to excavation and mitigate the possibility of spurious claims for excavation induced damages. Access to the properties for such surveys must be part of any encroachment agreements.

A careful evaluation of the subsurface soil conditions is required by the shoring designer to establish appropriate levels/elevations and design of the soil anchors. The anchor design will be governed by the weakest material in the profile. It is imperative that a detailed design is carried out at every different anchor level and location, and the anchors must be tested at each level.

Consideration should be given a post-grouted anchor system which may be a more feasible option for this site. The design adhesion for post-grouted earth anchors is controlled as much by the installation technique as the soil and therefore a proto-type anchor must be made and performance tested at each anchor level executed to demonstrate the anchor capacity and validate the design assumptions. This test must be completed before production anchors are made. Depending upon the location and elevation of the soil anchors, the post-grouted anchors at this site may carry an **ultimate** transfer load of about 70 to 90 kN/m made in clayey silt till of post-grouted anchor length depending upon the material type as confirmed by a performance/load test. It should be noted that these values are provided as preliminary guidance only and the actual anchor performance must be verified by a performance/load test.

Alternatively, rock anchors can be used. Anchors made in the inferred bedrock of the Georgian Bay Formation may be designed using a factored ULS adhesion of 620 kPa. Proto-type anchors must be performance tested to 200 percent of the design load to demonstrate the anchor capacity. All production anchors must be proof-tested to 133 percent of the design load, to validate the design assumptions.

Regardless, the subsurface soil information should be reviewed by the shoring designer to decide on the suitable type of earth anchors and design capacity values to be employed at this site.

If adjacent land owners are not agreeable to anchored support then internal bracing or rakers would be necessary. The footings for the rakers would be made in very stiff to hard clayey silt till where they could be designed for a bearing pressure of 150 kPa when inclined at 45 degrees.

5.10 Quality Control

Excavations on this site must be shored to preserve the integrity of the surrounding properties and structures. The Ontario Building Code 2012 stipulates that engineering review of the subsurface conditions is required on a continuous basis during the installation of earth retaining structures. Terraprobe should be retained to provide this review, which is an integral part of the geotechnical design function as it relates to the shoring design considerations. Terraprobe can provide detailed shoring design services for the project, if requested.

All foundations must be monitored by the geotechnical engineer on a continuous basis as they are constructed. The on-site review of the condition of the foundation soil as the foundations are constructed is an integral part of the geotechnical design function and is required by Section 4.2.2.2 of the Ontario Building Code 2012. If Terraprobe is not retained to carry out foundation evaluations during construction, then Terraprobe accepts no responsibility for the performance or non-performance of the foundations, even if they are ostensibly constructed in accordance with the conceptual design advice provided in this report.

Concrete for this structure will be specified in accordance with the requirements of CAN3 - CSA A23.1. Terraprobe maintains a CSA certified concrete laboratory and can provide concrete sampling and testing services for the project as necessary.

The requirements for fill placement on this project should be stipulated relative to Standard Proctor Maximum Dry Density (SPMDD), as determined by ASTM D698. In-situ determinations of density during fill placement by Procedure Method B of ASTM D2922 are recommended to demonstrate that the contractor is achieving the specified soil density. Terraprobe is a CNSC licensed operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary.

Terraprobe can provide thorough in house resources, quality control services for Building Envelope, Roofing, as well as Structural Steel in accordance with CSA W178, as necessary, for the Structural and Architectural quality control requirements of the project. Terraprobe is certified by the Canadian Welding Bureau under W178.1-1996.

6 LIMITATIONS AND RISK

6.1 Procedures

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained by Terraprobe.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities so that they may draw their own conclusions as to how the subsurface conditions may affect them.

6.2 Changes in Site and Scope

It must also be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The discussion and recommendations are based on the factual data obtained from this investigation conducted at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructability issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

This report was prepared for the express use of Brown Maple Investments Ltd. and their retained design consultants and is not for use by others. This report is copyright of Terraprobe Inc. and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe Inc. and Brown Maple Investments Ltd. who are the authorized users.

It is recognized that the regulatory agencies in their capacities as the planning and building authorities under Provincial statues, will make use of and rely upon this report, cognizant of the limitations thereof, both expressed and implied.

We trust the foregoing information is sufficient for your present requirements. If you have any questions, or if we can be of further assistance, please do not hesitate to contact us.

Yours truly,

Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
L ZHANG
100108585
20180808
PROVINCE OF ONTARIO

Seth Zhang, M. Eng., M. Sc., P. Eng.
Associate

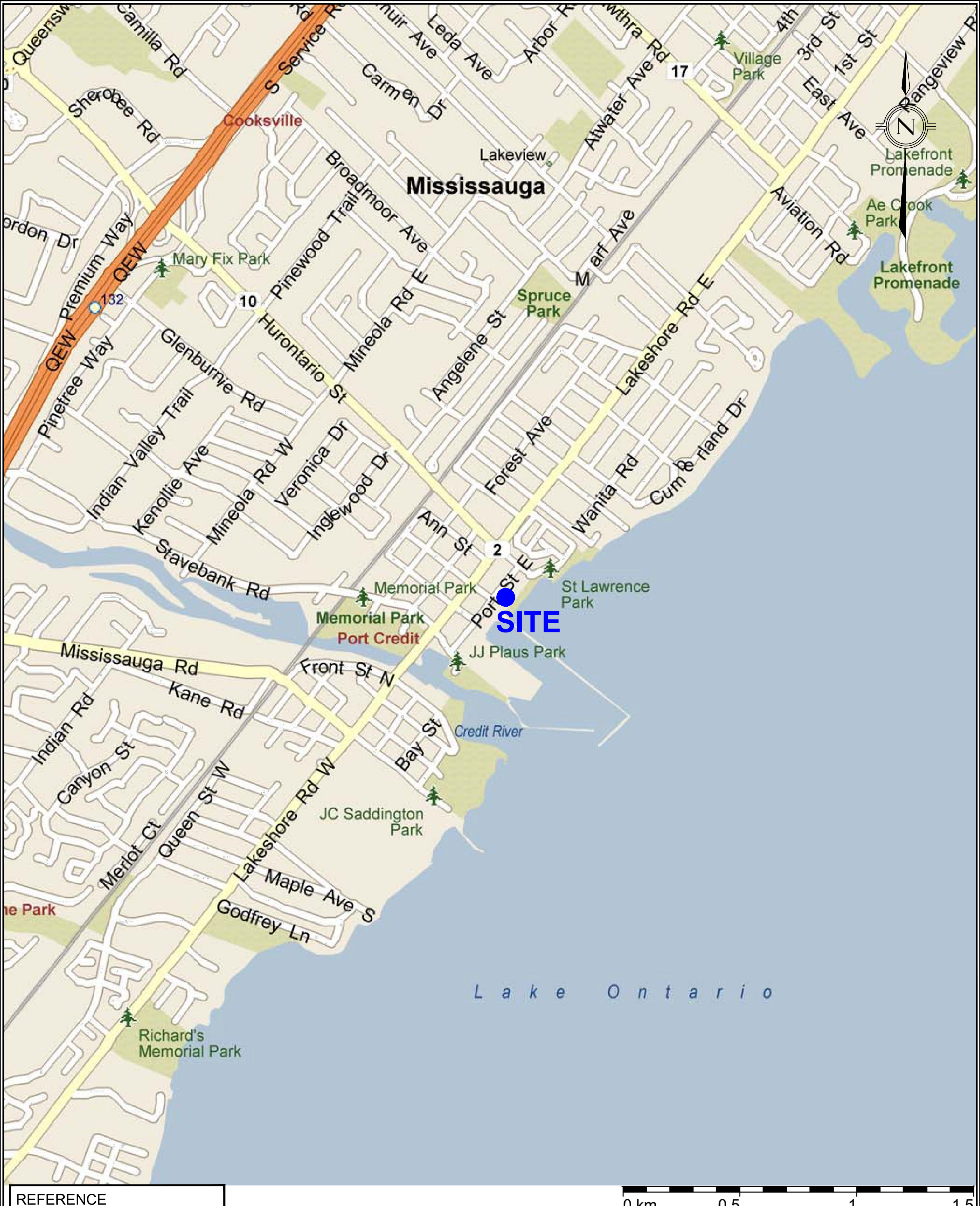
Mike Tanos, P. Eng.
Principal

ENCLOSURES

TERRAPROBE INC.

FIGURES

TERRAPROBE INC.



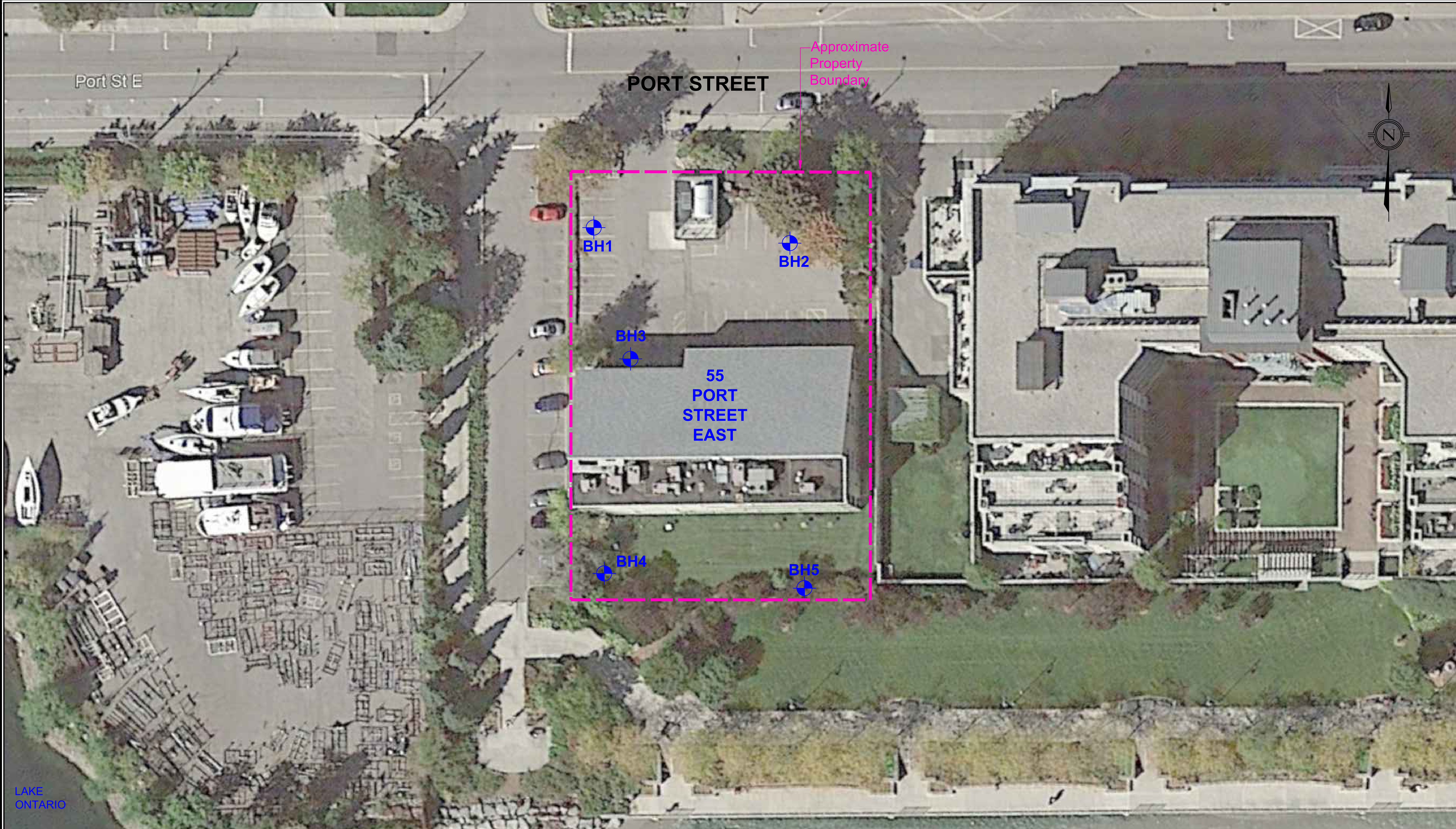
REFERENCE
Microsoft Streets & Trips 2012

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: SITE LOCATION PLAN

File No. 1-18-0012-01

FIGURE : 1



REFERENCE

City of Mississauga
emaps, aerial image 2017

LEGEND

Borehole Location

APPROXIMATE SCALE
10 8 6 4 2 0 10m

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:
BOREHOLE LOCATION AND SITE FEATURES PLAN
-EXISTING CONDITION

File No. 1-18-0012-01

FIGURE : 2A

REFERENCE
Drawing received via email from Giannone Petricone Associates Inc. Architects April 27, 2018.

LEGEND

Borehole Location

APPROXIMATE SCALE

10 8 6 4 2 0 10m

Terraprobe

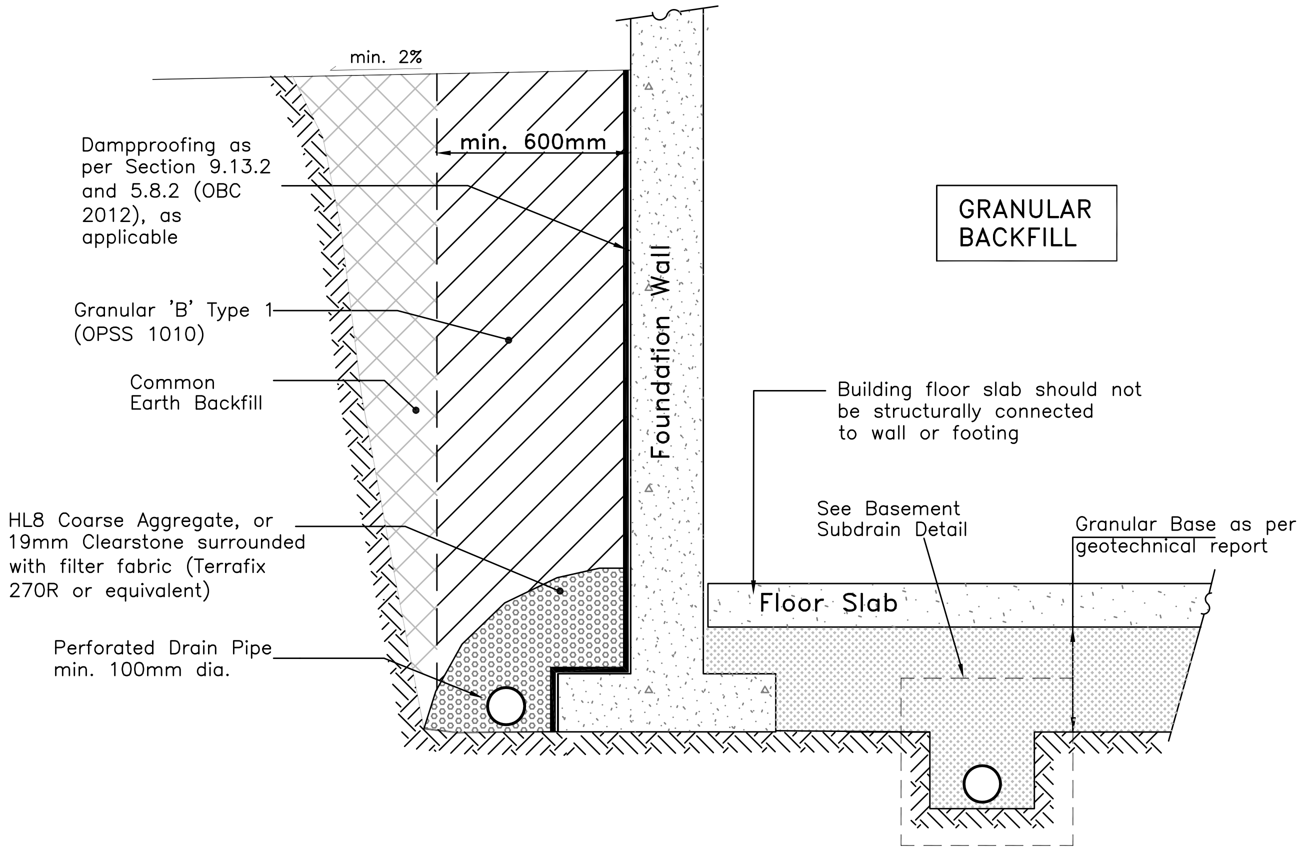
11 Indell Lane, Brampton, Ontario, L6T 3Y3

Tel: (905) 796-2650 Fax: (905) 796-2250

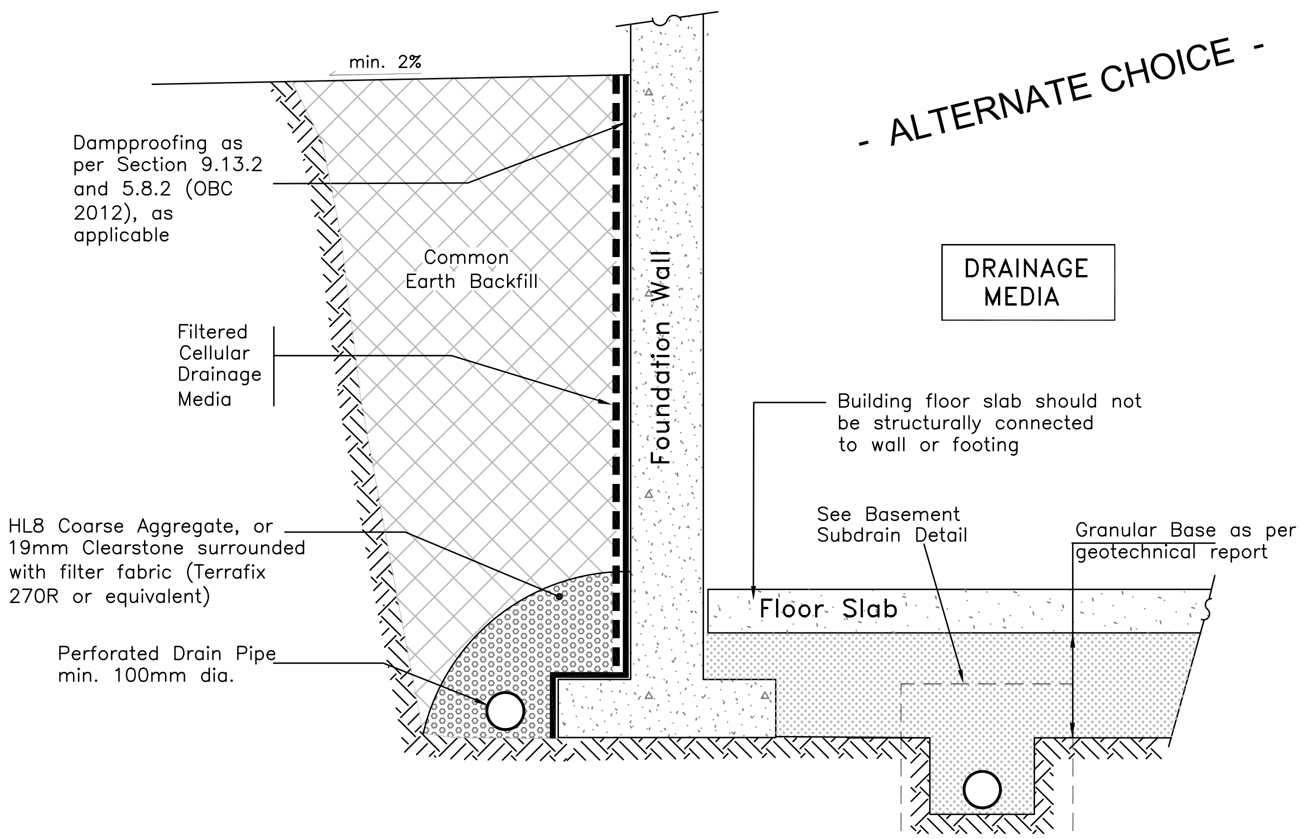
Title: BOREHOLE LOCATION AND SITE FEATURES PLAN -PROPOSED CONDITION

File No. 1-18-0012-01

FIGURE : 2B



- ALTERNATE CHOICE -



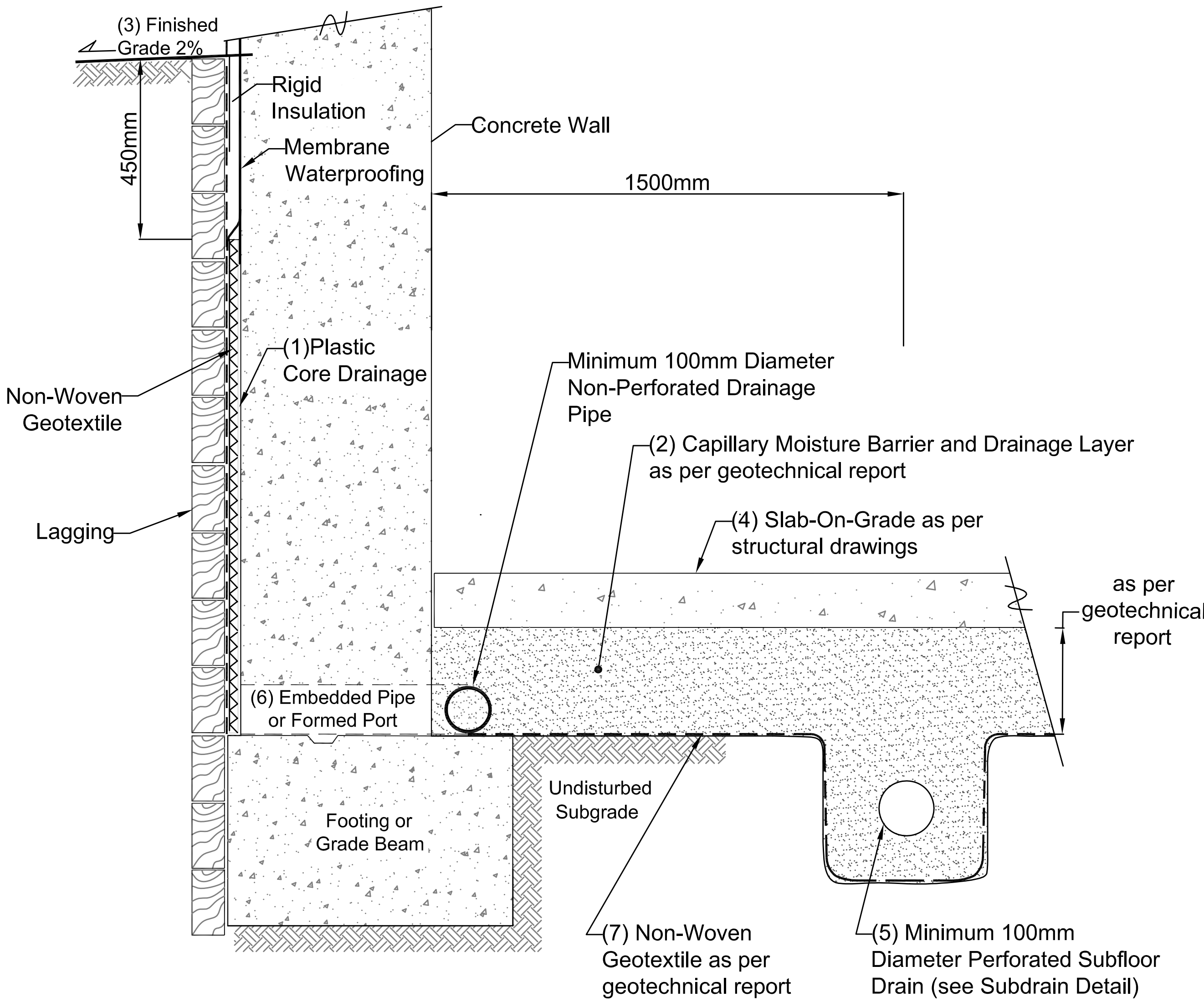
Schematic Only
Not to Scale

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title: BASEMENT DRAINAGE DETAIL

File No. 1-18-0012-01

FIGURE : 3



NOTES

1) Prefabricated drainage panels to consist of Terrafix - TERRADRAIN 200, Mirafi - Miradrain 6000, or approved equivalent. Panels should provide continuous cover with a minimum overlap of 300mm.

2) Capillary moisture barrier/drainage layer to consist of a minimum 200mm layer of 19mm clear stone (OPSS 1004), or as indicated in geotechnical report, compacted to a dense state. Upper 50mm can be replaced with Granular "A" (OPSS 1010) compacted to 98% SPMDD where vehicular traffic is required.

3) Exterior finished grade away from wall at a minimum grade of 2% for min. 1.2m.

4) Building floor slab-on-grade shall not be structurally connected to foundation wall or footing.

5) Subfloor drain invert to be a minimum of 300mm below underside of floor slab, to be set in parallel rows, one way, and at the spacing specified in the geotechnical report.

6) Embedded pipes/formed ports to be set a distance of maximum 3m on-centre. Each port to have a minimum cross-sectional area of 1500mm². Perimeter drainage must be collected and conveyed directly to the building sumps in non-perforated pipe.

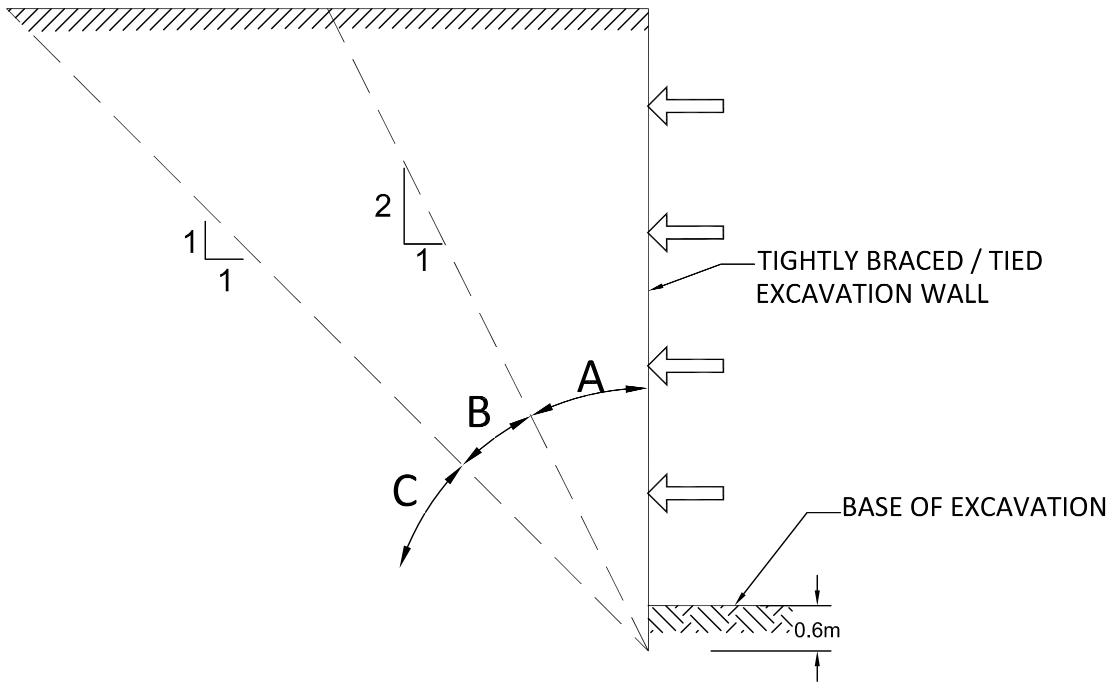
7) When the subgrade consists of a cohesionless soil, the subgrade must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).

N.T.S.

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

SCHEMATIC DRAINAGE DETAIL
SOLDIER PILE & LAGGING SHORING SYSTEM



Zone A: Foundations within this zone often require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone B: Foundation within this zone often do not require underpinning. Horizontal and vertical pressures on excavation wall of non-underpinned foundations must be considered.

Zone C: Foundations within this zone usually do not require underpinning.

REFERENCE:

User's Guide - NBC 2005 Structural Commentaries
(Part 4 of Division B) - Commentary K

Terraprobe
11 Indell Lane, Brampton, Ontario, L6T 3Y3
Tel: (905) 796-2650 Fax: (905) 796-2250

Title:

GUIDELINES FOR UNDERPINNING SOILS

APPENDIX A

TERRAPROBE INC.

Terraprobe

SAMPLING METHODS

AS	auger sample
CORE	cored sample
DP	direct push
FV	field vane
GS	grab sample
SS	split spoon
ST	shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Test (SPT) resistance ('N' values) is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a standard 50 mm (2 in.) diameter split spoon sampler for a distance of 0.3 m (12 in.).

Dynamic Cone Test (DCT) resistance is defined as the number of blows by a hammer weighing 63.6 kg (140 lb.) falling freely for a distance of 0.76 m (30 in.) required to advance a conical steel point of 50 mm (2 in.) diameter and with 60° sides on 'A' size drill rods for a distance of 0.3 m (12 in.)."

COHESIONLESS SOILS

Compactness	'N' value
very loose	< 4
loose	4 – 10
compact	10 – 30
dense	30 – 50
very dense	> 50

COHESIVE SOILS

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

COMPOSITION

Term (e.g)	% by weight
trace silt	< 10
some silt	10 – 20
silty	20 – 35
sand *and* silt	> 35

TESTS AND SYMBOLS

MH	mechanical sieve and hydrometer analysis
w, w_c	water content
w_L, LL	liquid limit
w_P, PL	plastic limit
I_P, PI	plasticity index
k	coefficient of permeability
γ	soil unit weight, bulk
G_s	specific gravity
φ'	internal friction angle
c'	effective cohesion
c_u	undrained shear strength

▽	Unstabilized water level
▼	1st water level measurement
▼	2nd water level measurement
▼	Most recent water level measurement
3.0 +	Undrained shear strength from field vane (with sensitivity)
C_c	compression index
c_v	coefficient of consolidation
m_v	coefficient of compressibility
e	void ratio

FIELD MOISTURE DESCRIPTIONS

Damp	refers to a soil sample that does not exhibit any observable pore water from field/hand inspection.
Moist	refers to a soil sample that exhibits evidence of existing pore water (e.g. sample feels cool, cohesive soil is at or close to plastic limit) but does not have visible pore water
Wet	refers to a soil sample that has visible pore water

Terraprobe

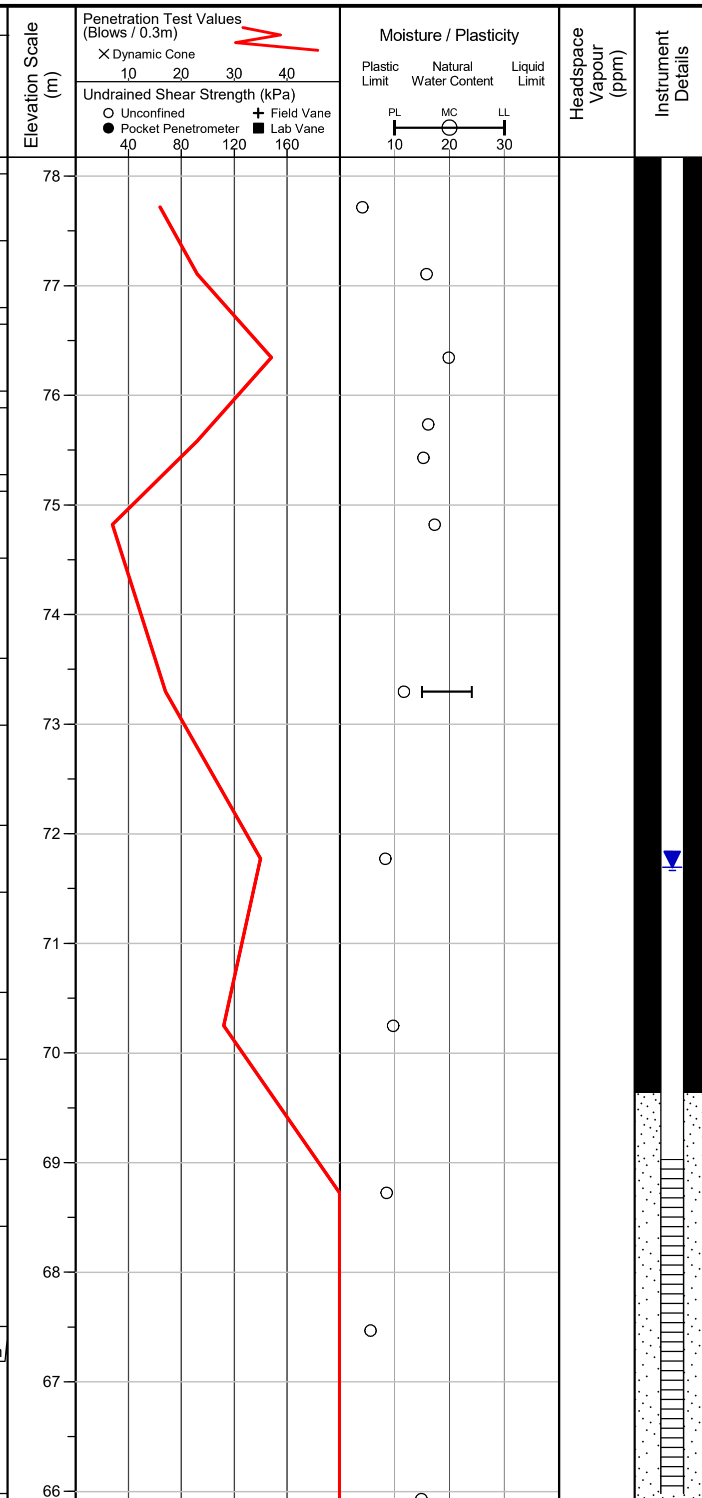
LOG OF BOREHOLE 1

Project No. : 1-18-0012-01	Client : Brown Maple Investments Ltd	Originated by : BR
Date started : March 21, 2018	Project : 55 Port Street East	Compiled by : SZ
Sheet No. : 1 of 1	Location : Mississauga, Ontario	Checked by : BS

Position : E: 614557, N: 4823234 (UTM 17T) — Elevation Datum : Geodetic

Rig type : Track-mounted — Drilling Method : Solid stem augers

Elev / Depth (m)	Description	Sample Number	Type	SPT 'N' Value	Lab Data and Comments (Grain Size Distribution (%) (MIT) GR SA SI CL)
78.2	GROUND SURFACE				
	80mm **ASPHALTIC CONCRETE**				
	670mm **AGGREGATE**	1	SS	16	
77.4 / 0.8	**SANDY SILT to SILT AND SAND**, trace gravel, trace clay, compact to dense, grey, moist to wet	2	SS	23	
		3	SS	37	
		4A	SS	23	
75.6 / 2.6	**SILT**, some clay, trace to some sand, dilatant, loose to compact, grey, moist	4B			0 7 75 18
		5	SS	7	
73.6 / 4.6	**CLAYEY SILT**, some sand to sandy, trace to some gravel, trace shale fragments, very stiff to hard, grey, moist (GLACIAL TILL)	6	SS	17	12 28 39 21
		7	SS	35	
		8	SS	28	
		9	SS	66	
67.5 / 10.7	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	10	SS	50 / 75mm	
65.9 / 12.3		11	SS	70 / 100mm	



END OF BOREHOLE

Unstabilized water level measured at 10.4 m below ground surface; borehole was open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Apr 2, 2018	6.5	71.7

file: 1-18-0012-01 bh logs.gpj

LOG OF BOREHOLE 2

Terraprobe

Project No. : 1-18-0012-01
Client : Brown Maple Investments Ltd
Originated by : BR

Date started : March 21, 2018
Project : 55 Port Street East
Compiled by : SZ

Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : BS

Position : E: 614576, N: 4823248 (UTM 17T)
Elevation Datum : Geodetic

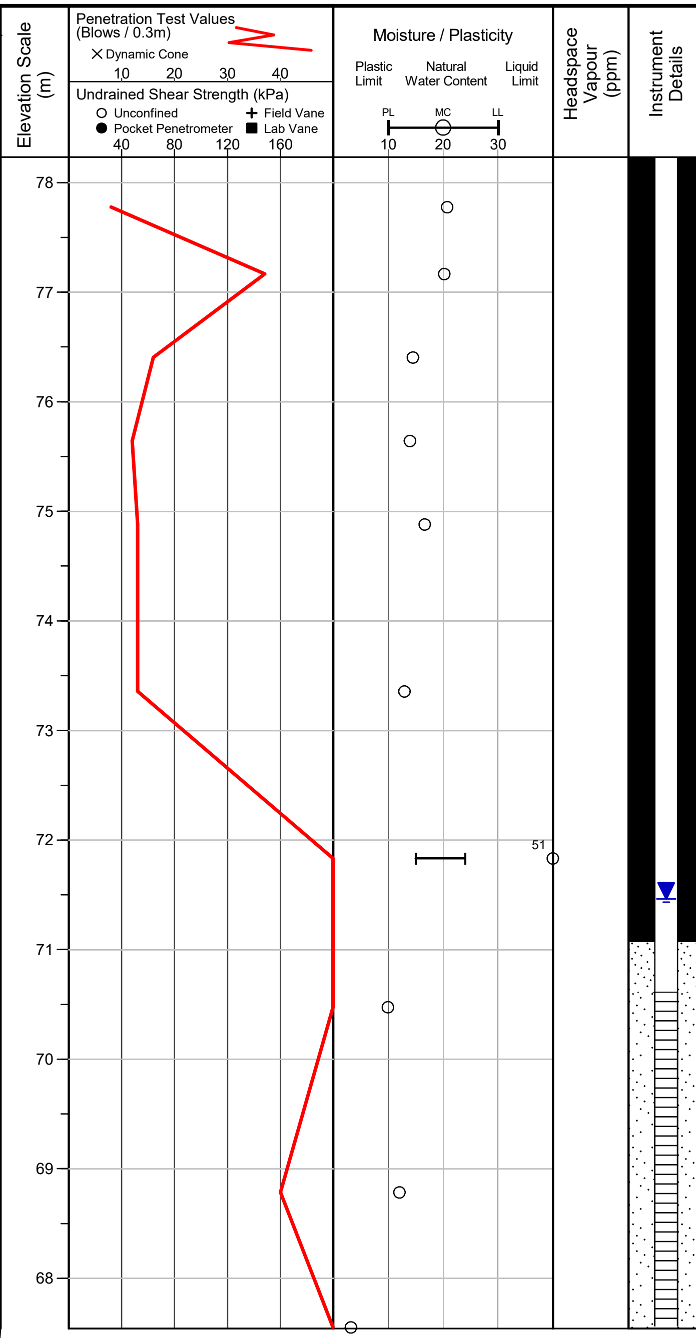
Rig type : Track-mounted
Drilling Method : Solid stem augers

Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value	Lab Data and Comments: GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
78.2	**GROUND SURFACE**				
78.0 / 0.2	90mm **ASPHALTIC CONCRETE**; 140mm **AGGREGATE**				
	SANDY SILT to SILT AND SAND, trace gravel, trace clay, loose to dense, brown, moist to wet	1	SS	8	
		2	SS	37	0 43 53 4
		3	SS	16	
75.9 / 2.3	**SILT**, some clay, trace to some sand, dilatant, compact, grey, moist	4	SS	12	
		5	SS	13	
73.6 / 4.6	**CLAYEY SILT**, some sand to sandy, trace to some gravel, trace shale fragments, stiff, grey, moist (GLACIAL TILL)	6	SS	13	
	...hard below	7	SS	76	10 28 40 22
		8	SS	50 / 125mm	
		9	SS	40	
67.5 / 67.5 / 10.7	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	10	SS	50 / 25mm	auger grinding

END OF BOREHOLE

Unstabilized water level measured at 10.5 m below ground surface; borehole was open upon completion of drilling.

50 mm dia. monitoring well installed.



WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Apr 2, 2018	6.8	71.5

file: 1-18-0012-01 bh logs.gpj

Terraprobe

LOG OF BOREHOLE 3

Project No. : 1-18-0012-01	Client : Brown Maple Investments Ltd	Originated by : BR
Date started : March 22, 2018	Project : 55 Port Street East	Compiled by : SZ
Sheet No. : 1 of 1	Location : Mississauga, Ontario	Checked by : BS

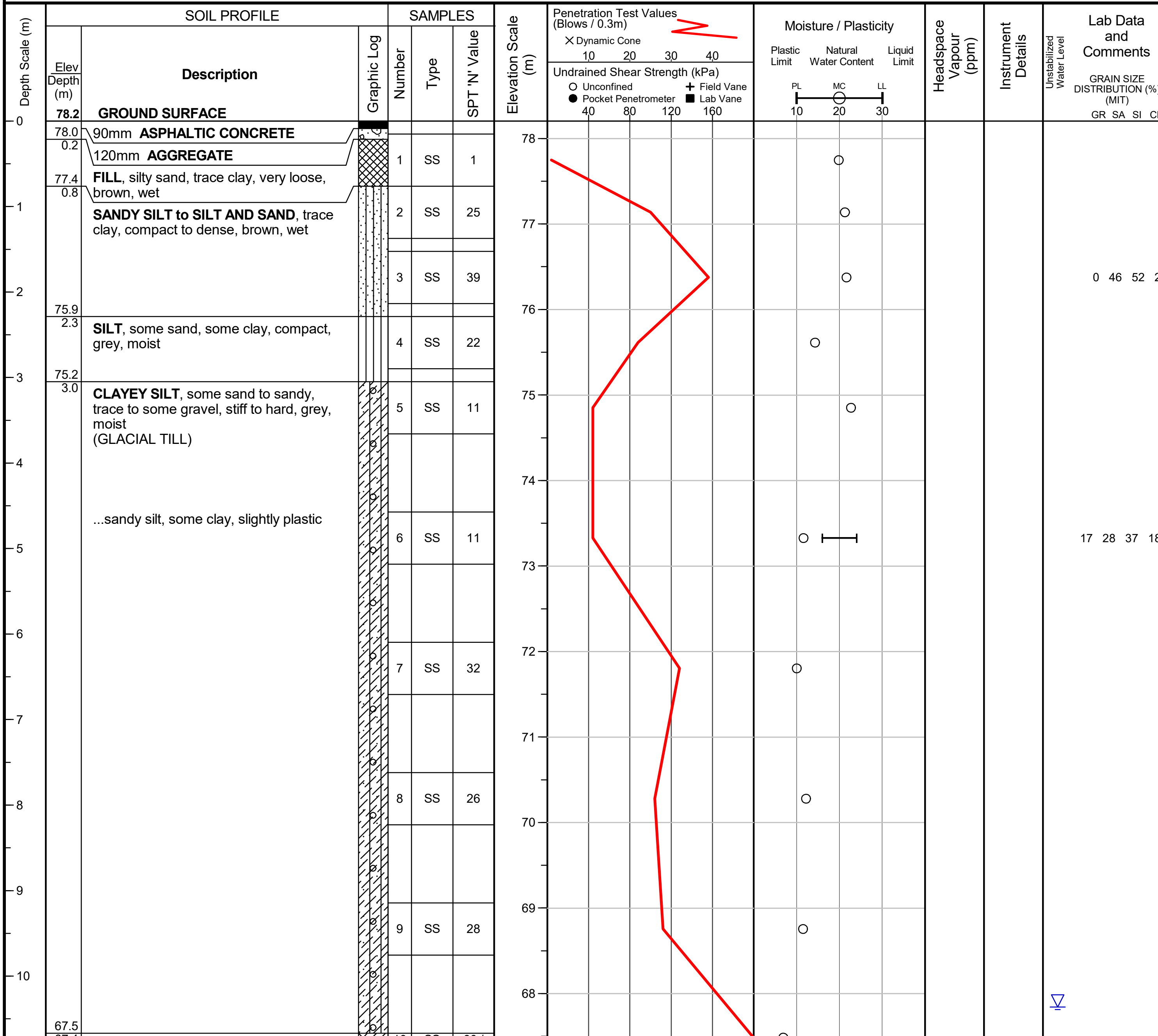
Position : E: 614576, N: 4823222 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Track-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev (m)	Depth (m)	Description
78.2		**GROUND SURFACE**
78.0	0.2	90mm **ASPHALTIC CONCRETE**
		120mm **AGGREGATE**
77.4	0.8	**FILL**, silty sand, trace clay, very loose, brown, wet
		SANDY SILT to SILT AND SAND, trace clay, compact to dense, brown, wet
75.9	2.3	**SILT**, some sand, some clay, compact, grey, moist
75.2	3.0	**CLAYEY SILT**, some sand to sandy, trace to some gravel, stiff to hard, grey, moist (GLACIAL TILL) ...sandy silt, some clay, slightly plastic
67.5 / 67.4	10.8	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)

SAMPLES

Number	Type	SPT 'N' Value	Lab Data and Comments: GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
1	SS	1	
2	SS	25	
3	SS	39	0 46 52 2
4	SS	22	
5	SS	11	
6	SS	11	17 28 37 18
7	SS	32	
8	SS	26	
9	SS	28	
10	SS	60 / 100mm	



END OF BOREHOLE

Unstabilized water level measured at 10.4 m below ground surface; borehole was open upon completion of drilling.

file: 1-18-0012-01 bh logs.gpj

Terraprobe

LOG OF BOREHOLE 4

Project No. : 1-18-0012-01 | Client : Brown Maple Investments Ltd | Originated by : BR

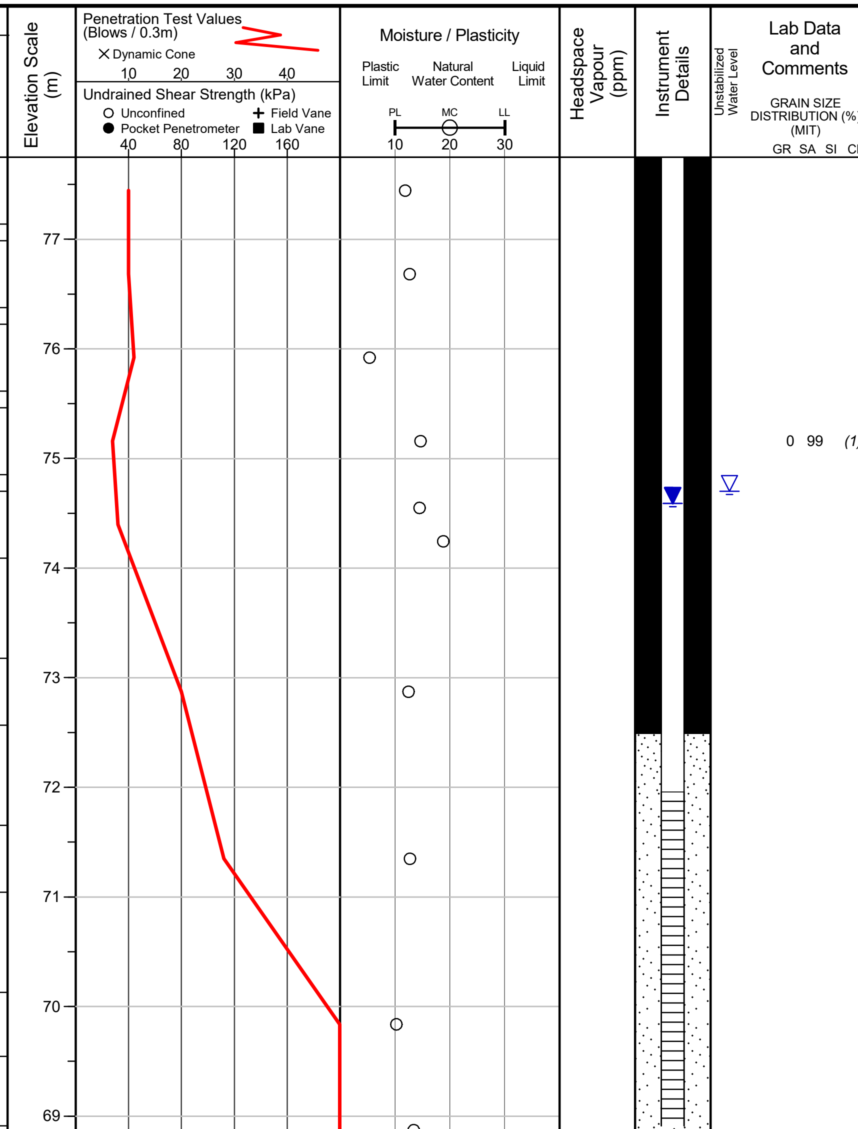
Date started : March 23, 2018 | Project : 55 Port Street East | Compiled by : SZ

Sheet No. : 1 of 1 | Location : Mississauga, Ontario | Checked by : BS

Position : E: 614594, N: 4823208 (UTM 17T) | Elevation Datum : Geodetic

Rig type : Track-mounted | Drilling Method : Solid stem augers

Elev / Depth (m)	Description (SOIL PROFILE)	Sample Number	Type	SPT 'N' Value	Lab Data and Comments (GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL)
77.7	**GROUND SURFACE**				
	100mm **TOPSOIL**				
	FILL, clayey silt, some sand, trace gravel, trace construction debris, stiff, brown, moist	1	SS	10	
		2	SS	10	
76.2 / 1.5	**FILL**, sand, trace silt, trace gravel, compact, dark brown, moist	3	SS	11	
75.4 / 2.3	**SAND**, trace silt, loose, brown, wet	4	SS	7	0 99 (1)
74.7 / 3.0	**SILT**, some sand, some clay, loose, grey, wet	5A	SS	8	
74.3 / 3.4	**CLAYEY SILT**, some sand to sandy, trace to some gravel, firm to very stiff, grey, moist (GLACIAL TILL)	5B			
		6	SS	20	
		7	SS	28	
	...hard below	8	SS	73	
68.9		9	SS	60 / 75mm	
68.8 / 8.9	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)				auger refusal



END OF BOREHOLE
Auger refusal

Unstabilized water level measured at 3.0 m below ground surface; borehole was open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Apr 2, 2018	3.2	74.6

Terraprobe

LOG OF BOREHOLE 5

Project No. : 1-18-0012-01 | Client : Brown Maple Investments Ltd | Originated by : BR

Date started : March 22, 2018 | Project : 55 Port Street East | Compiled by : SZ

Sheet No. : 1 of 1 | Location : Mississauga, Ontario | Checked by : BS

Position : E: 614616, N: 4823227 (UTM 17T) | Elevation Datum : Geodetic

Rig type : Track-mounted | Drilling Method : Solid stem augers

Elev / Depth (m)	Description	Sample Number	Type	SPT 'N' Value	Lab Data and Comments
77.6	**GROUND SURFACE**				
	100mm **TOPSOIL**				
	FILL, clayey silt, some sand, trace gravel, trace construction debris, stiff, brown, moist	1	SS	12	
76.8 / 0.8	**FILL**, sand, trace silt, trace gravel, trace organics, loose to compact, brown, moist	2	SS	14	
		3	SS	8	
75.3 / 2.3	**SAND**, trace silt, loose, brown, wet	4	SS	10	
74.6 / 3.0	**SANDY GRAVEL**, trace silt, dense, grey, wet	5	SS	41	spoon wet
73.0 / 4.6	**CLAYEY SILT**, some sand to sandy, trace to some gravel, trace shale fragments, stiff, grey, moist (GLACIAL TILL)	6	SS	9	
	...hard below	7	SS	83	GR 10, SA 26, SI 41, CL 23
		8	SS	33	
		9	SS	76 / 275mm	
66.9 / 66.9 / 10.7	**INFERRED BEDROCK** (GEORGIAN BAY FORMATION)	10	SS	60 / 75mm	auger grinding, refusal

END OF BOREHOLE

Unstabilized water level measured at 3.0 m below ground surface; borehole caved to 4.6 m below ground surface upon completion of drilling.

50 mm dia. monitoring well installed.

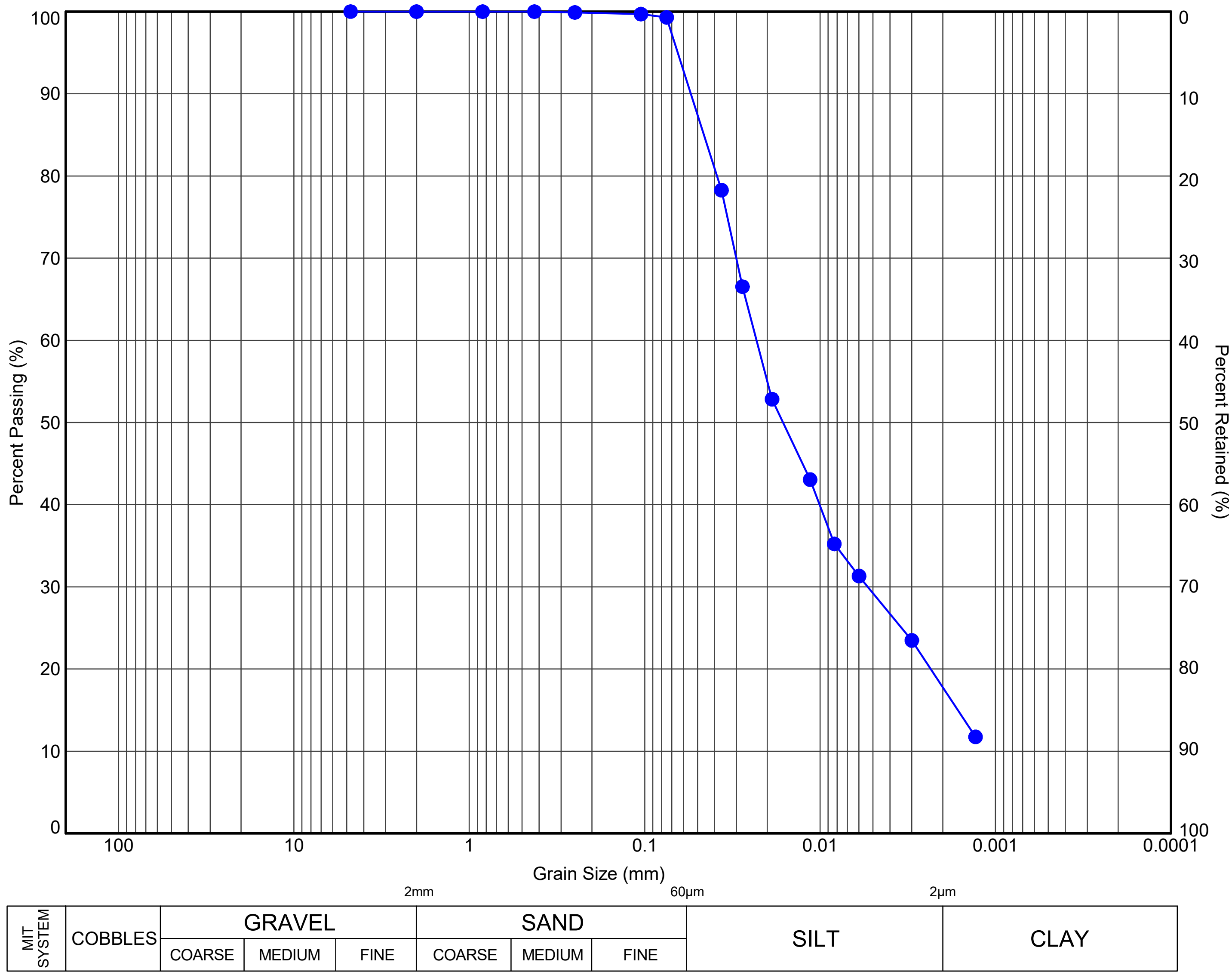
WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Apr 2, 2018	3.0	74.7

file: 1-18-0012-01 bh logs.gpj

APPENDIX B

TERRAPROBE INC.



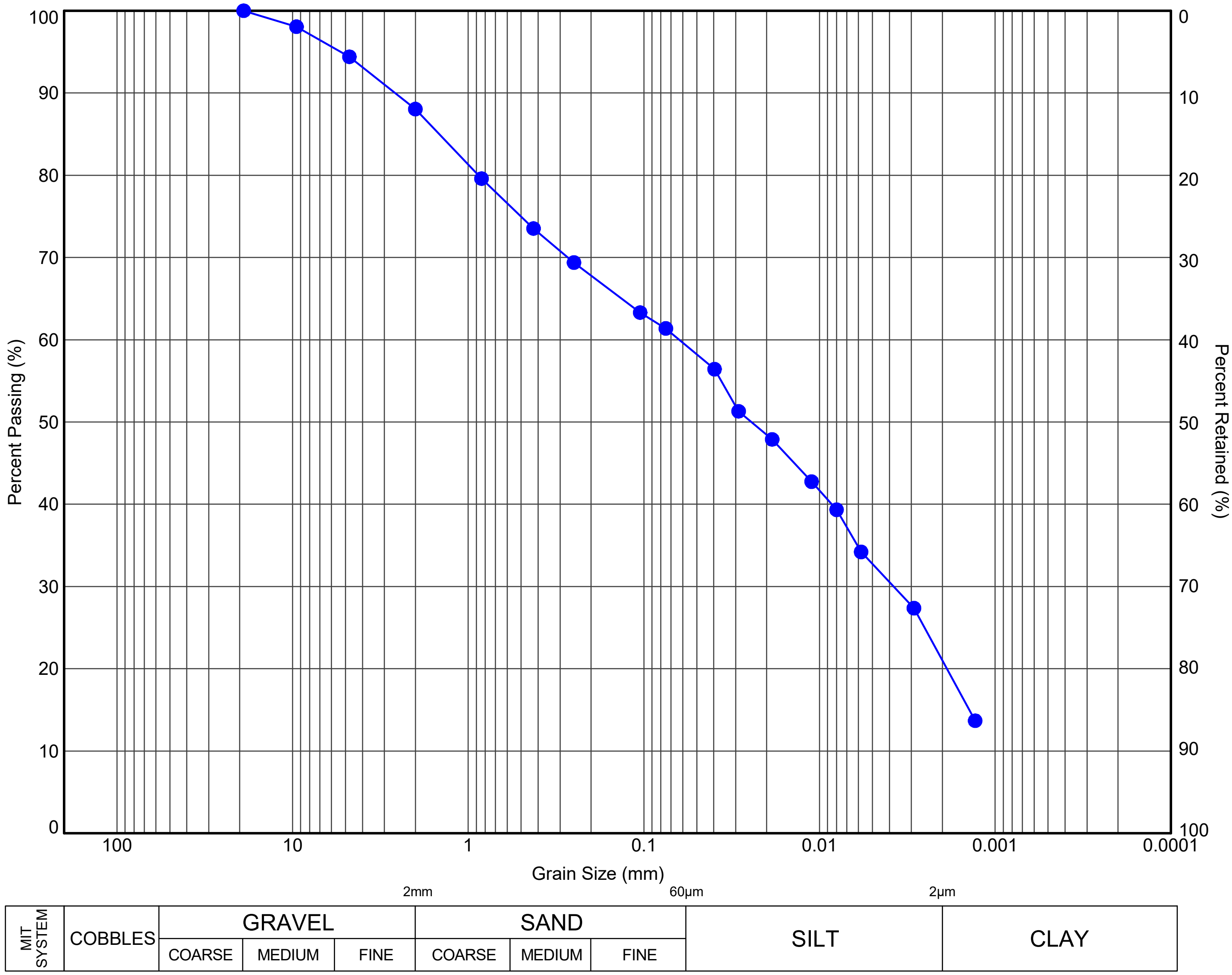
MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**1**	4B	2.7	75.4	0	7	75	18	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
SILT, SOME CLAY, TRACE SAND

File No.: **1-18-0012-01**



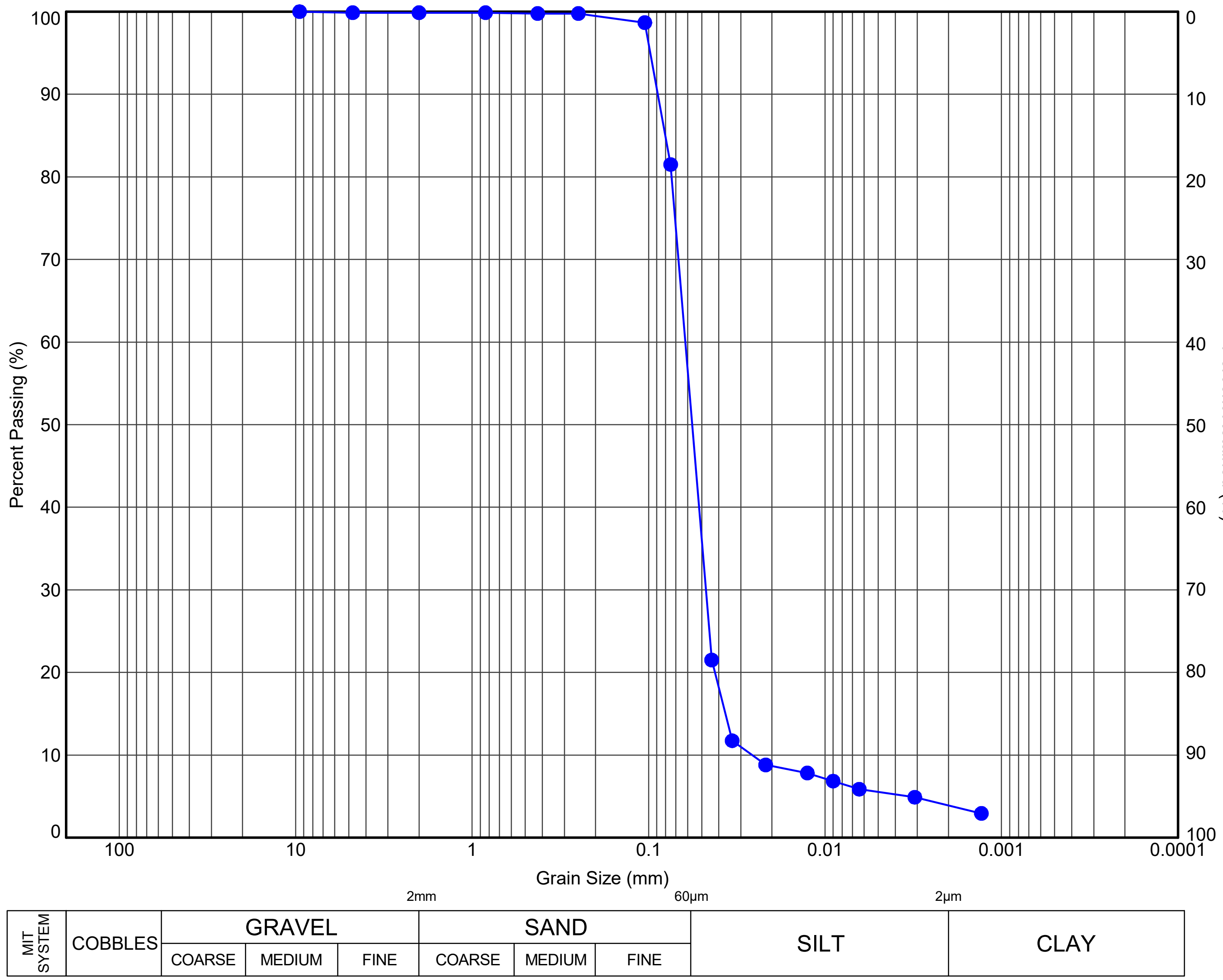
MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**1**	SS6	4.9	73.3	12	28	39	21	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
CLAYEY SILT, SANDY, SOME GRAVEL

File No.: **1-18-0012-01**



MIT SYSTEM

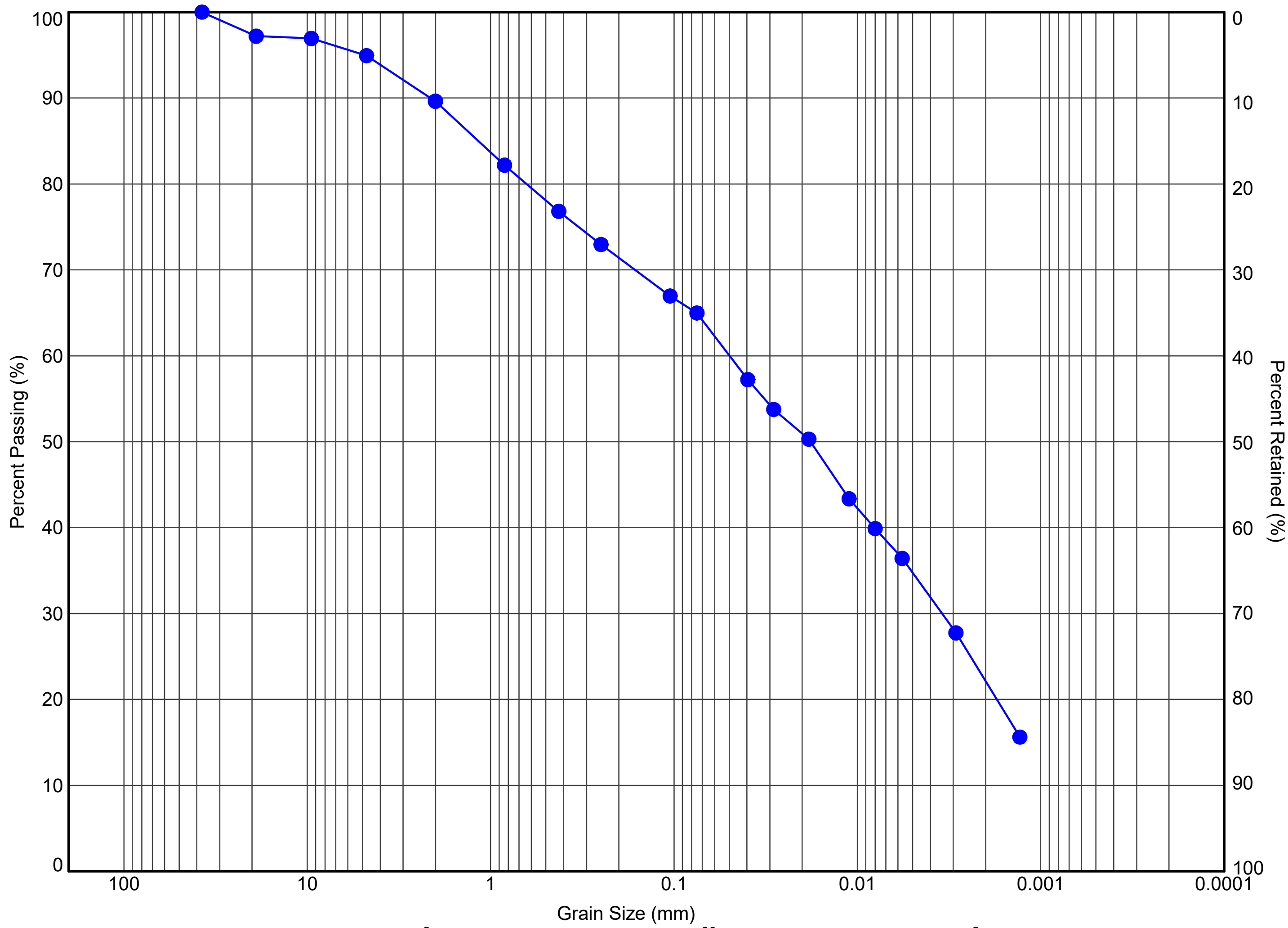
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**2**	SS2	1.1	77.2	0	43	53	4	

Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
SILT AND SAND, TRACE CLAY

File No.: **1-18-0012-01**



	2mm			60µm				2µm		
MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

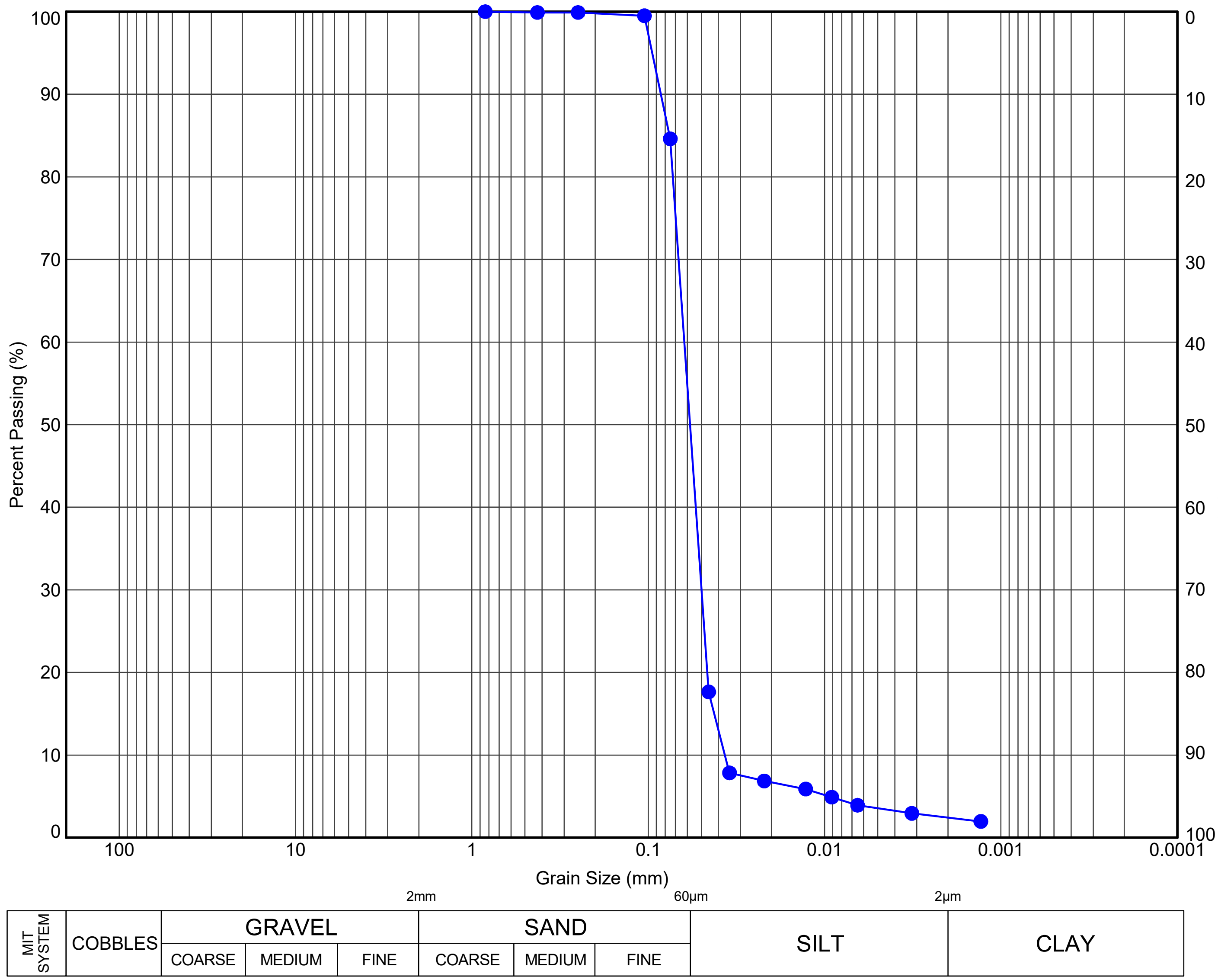
MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**2**	SS7	6.4	71.8	10	28	40	22	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
CLAYEY SILT, SANDY, TRACE GRAVEL

File No.: **1-18-0012-01**



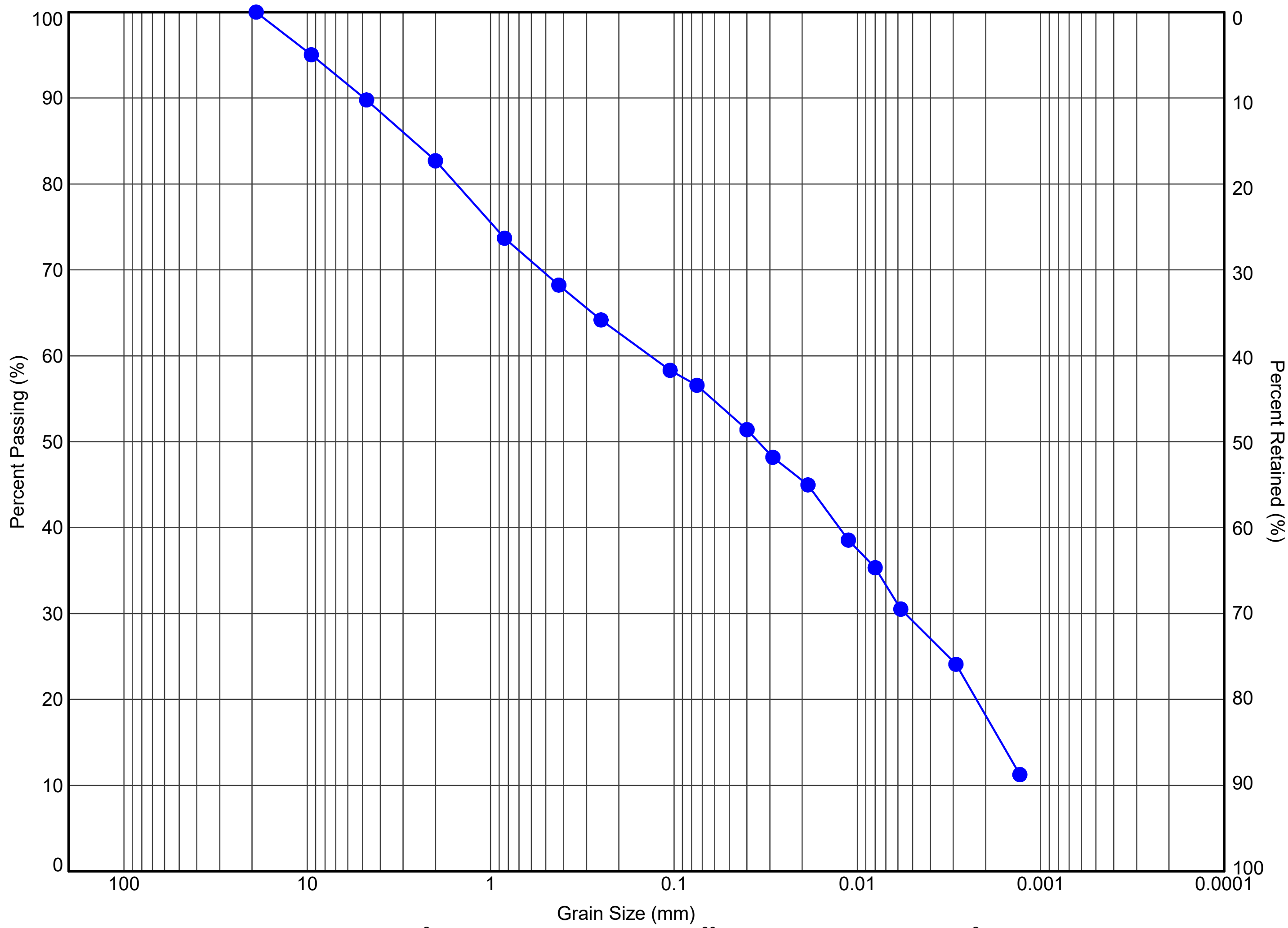
				MIT SYSTEM				
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
● **3**	SS3	1.8	76.4	0	46	52	2	

Terraprobe

11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
SILT AND SAND, TRACE CLAY

File No.: **1-18-0012-01**



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

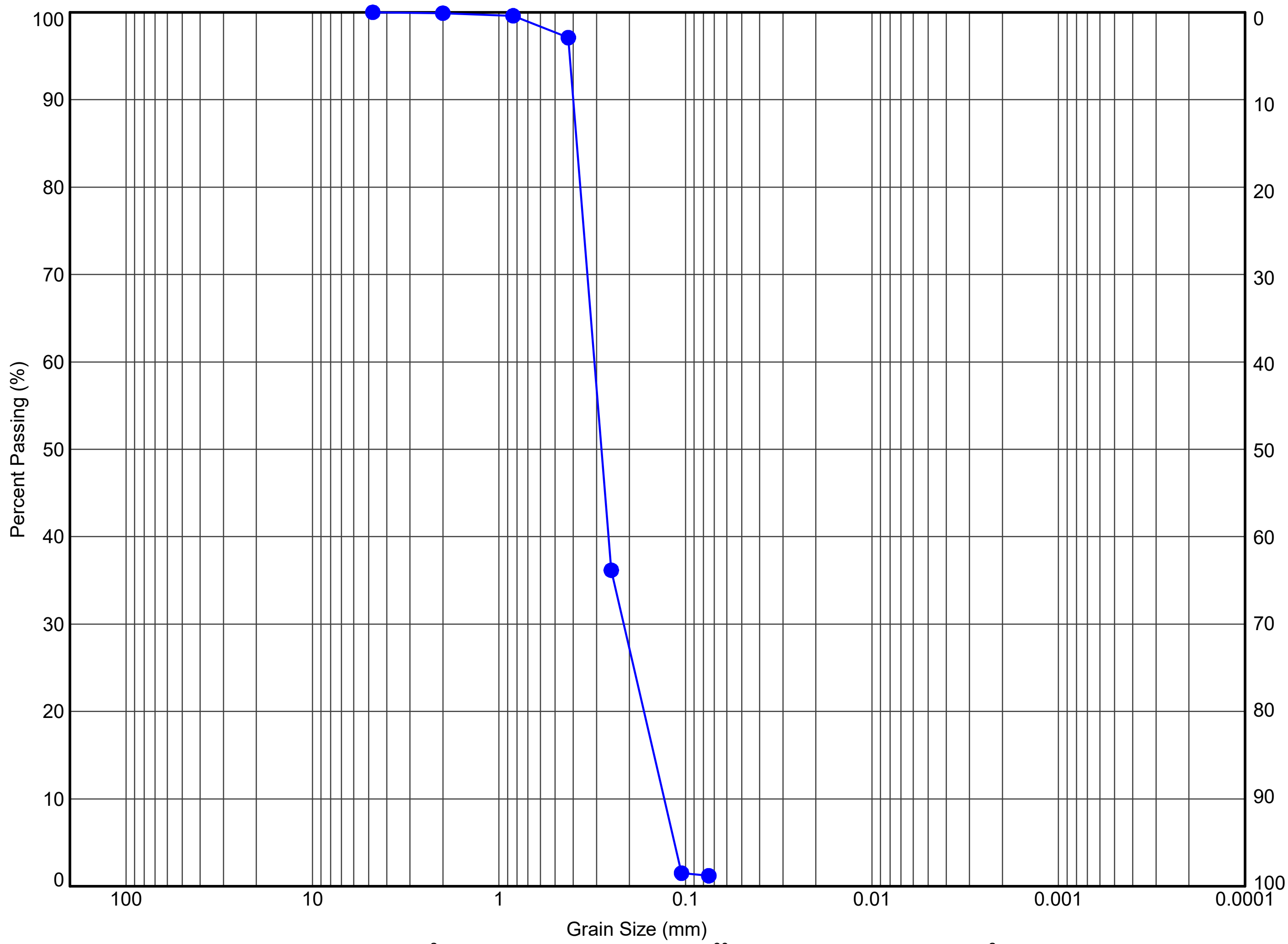
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
					MIT SYSTEM				
●	**3**	SS6	4.9	73.3	17	28	37	18	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
SANDY SILT, SOME CLAY, SOME GRAVEL

File No.: **1-18-0012-01**



2mm 60μm 2μm

MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

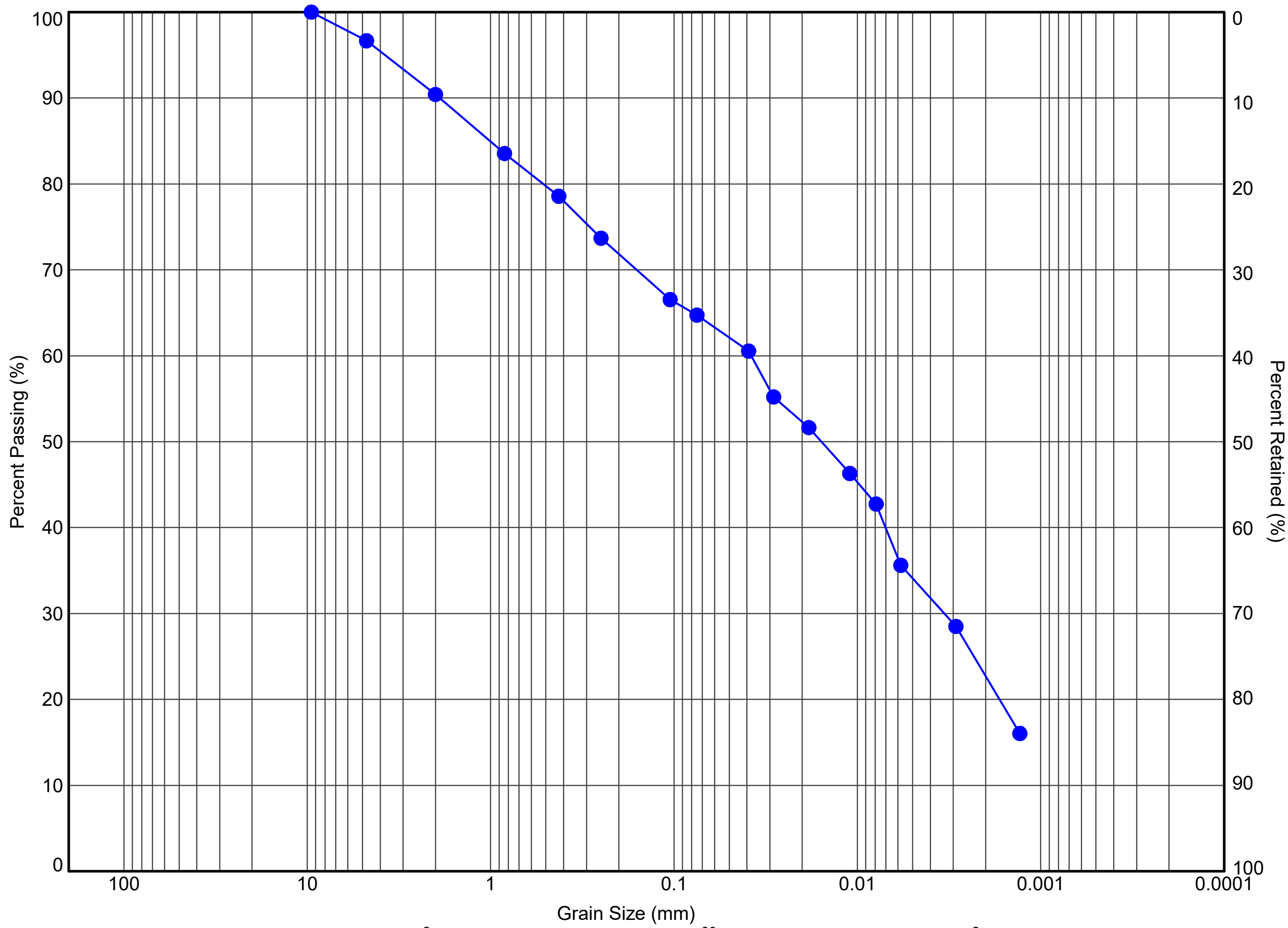
MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**4**	SS4	2.6	75.2	0	99			*(1)*

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title: **GRAIN SIZE DISTRIBUTION**
SAND, TRACE SILT

File No.: **1-18-0012-01**



MIT SYSTEM	COBBLES	GRAVEL			SAND			SILT	CLAY
		COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE		

2mm 60µm 2µm

MIT SYSTEM

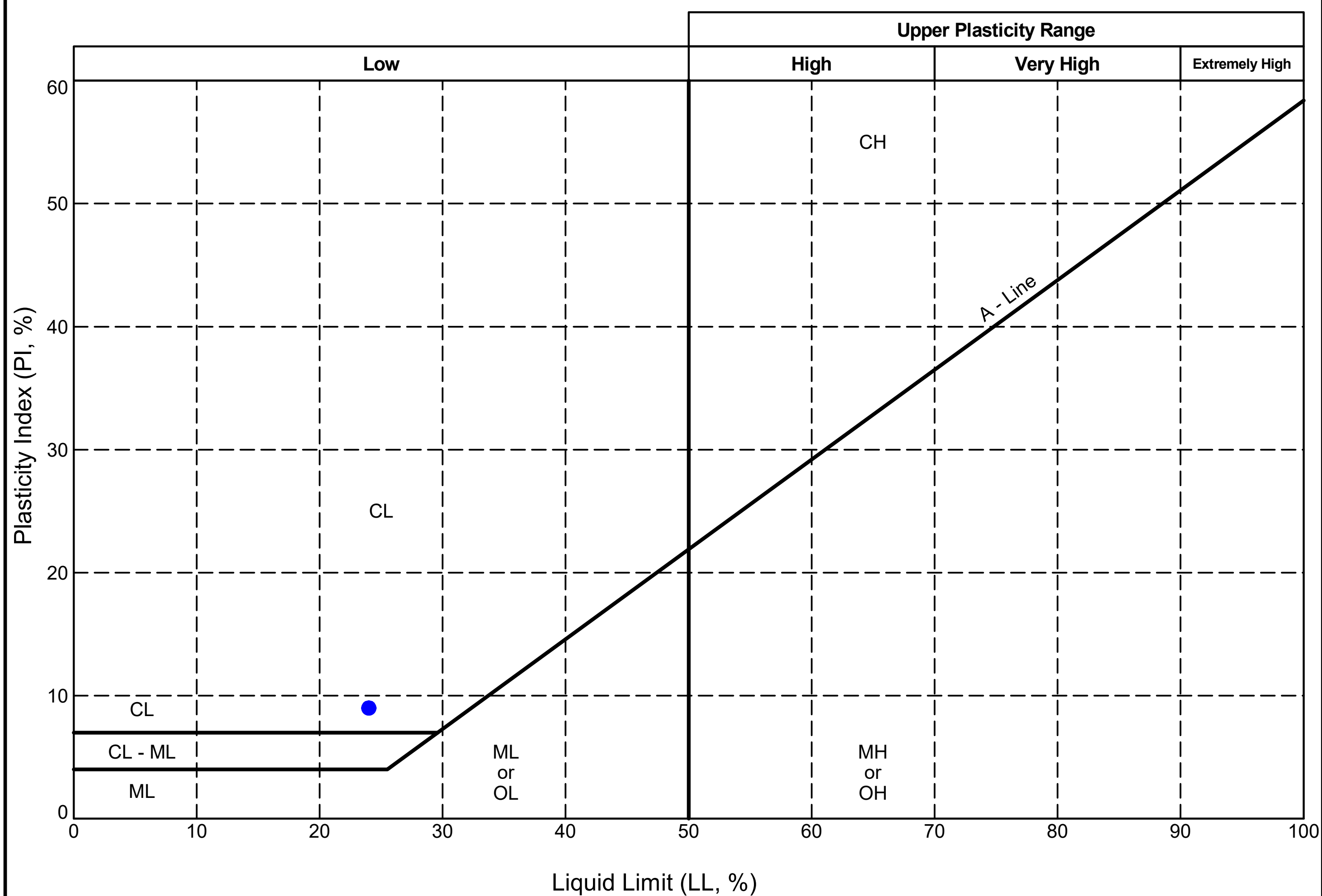
	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**5**	SS7	6.4	71.2	10	26	41	23	

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SANDY, TRACE GRAVEL

File No.: **1-18-0012-01**



	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)	Description
●	1	SS6	4.9	73.3	24	15	9	SLIGHTLY PLASTIC

Terraprobe

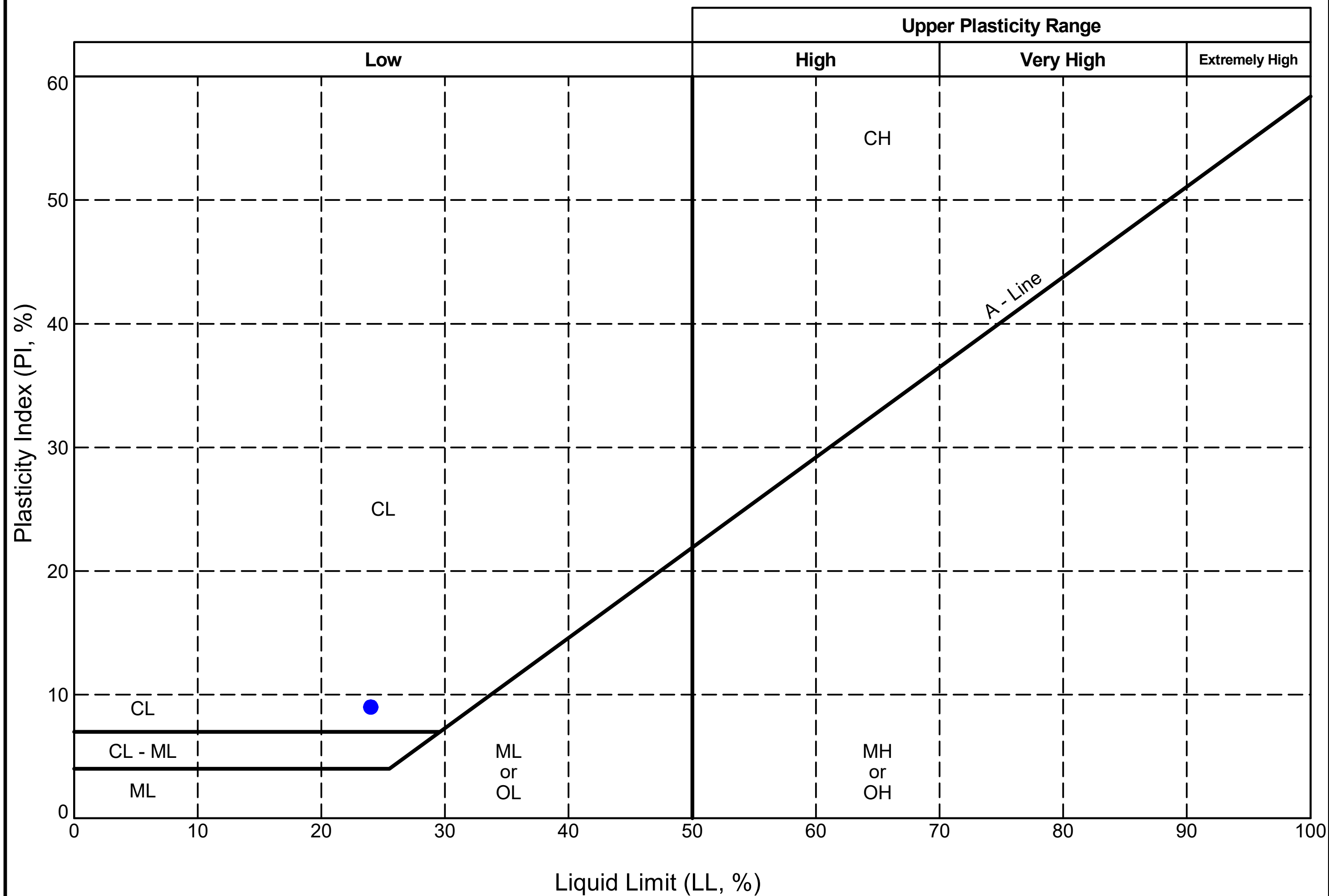
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

ATTERBERG LIMITS CHART

File No.:

1-18-0012-01



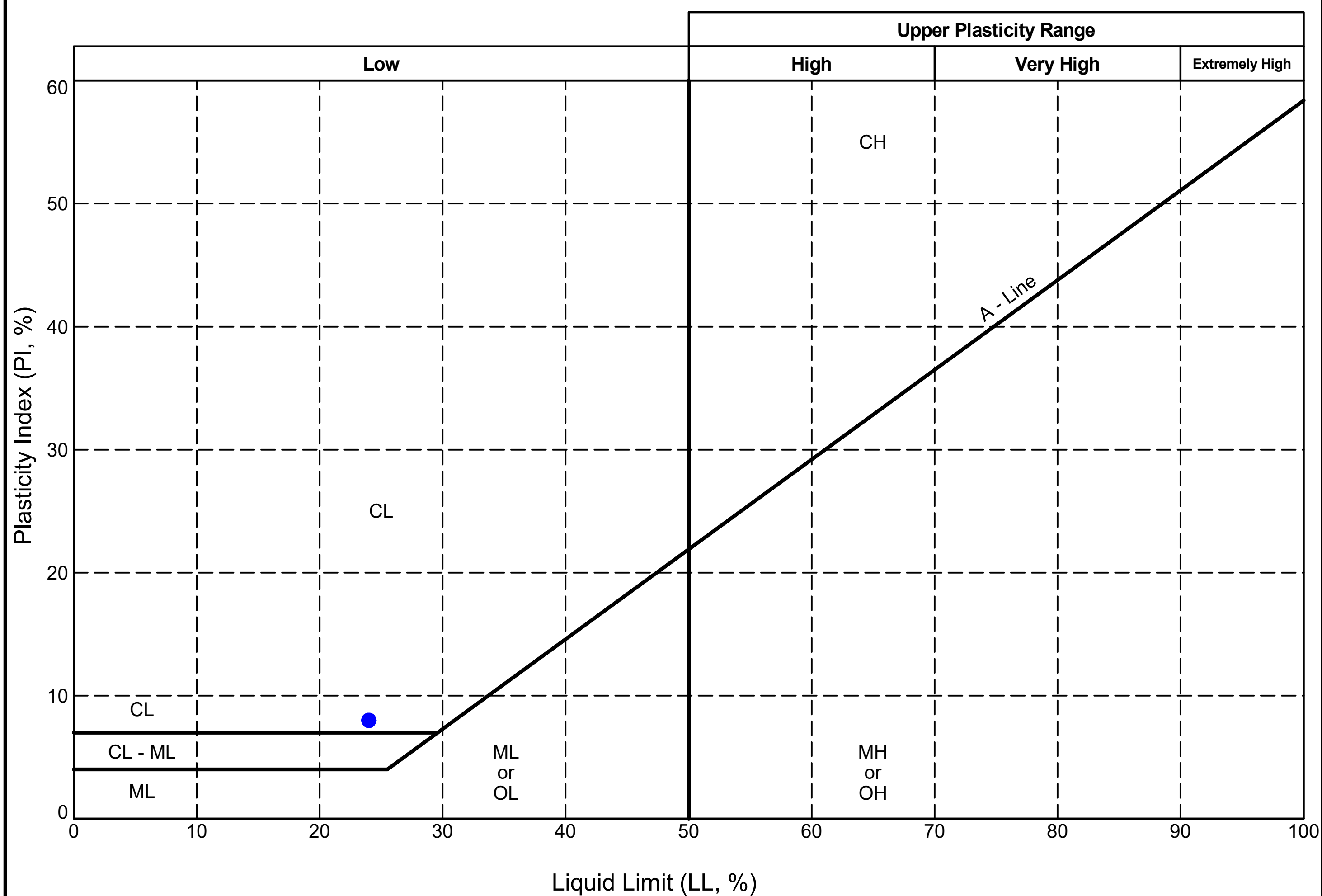
	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)	Description
●	2	SS7	6.4	71.8	24	15	9	SLIGHTLY PLASTIC

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

ATTERBERG LIMITS CHART

File No.: **1-18-0012-01**



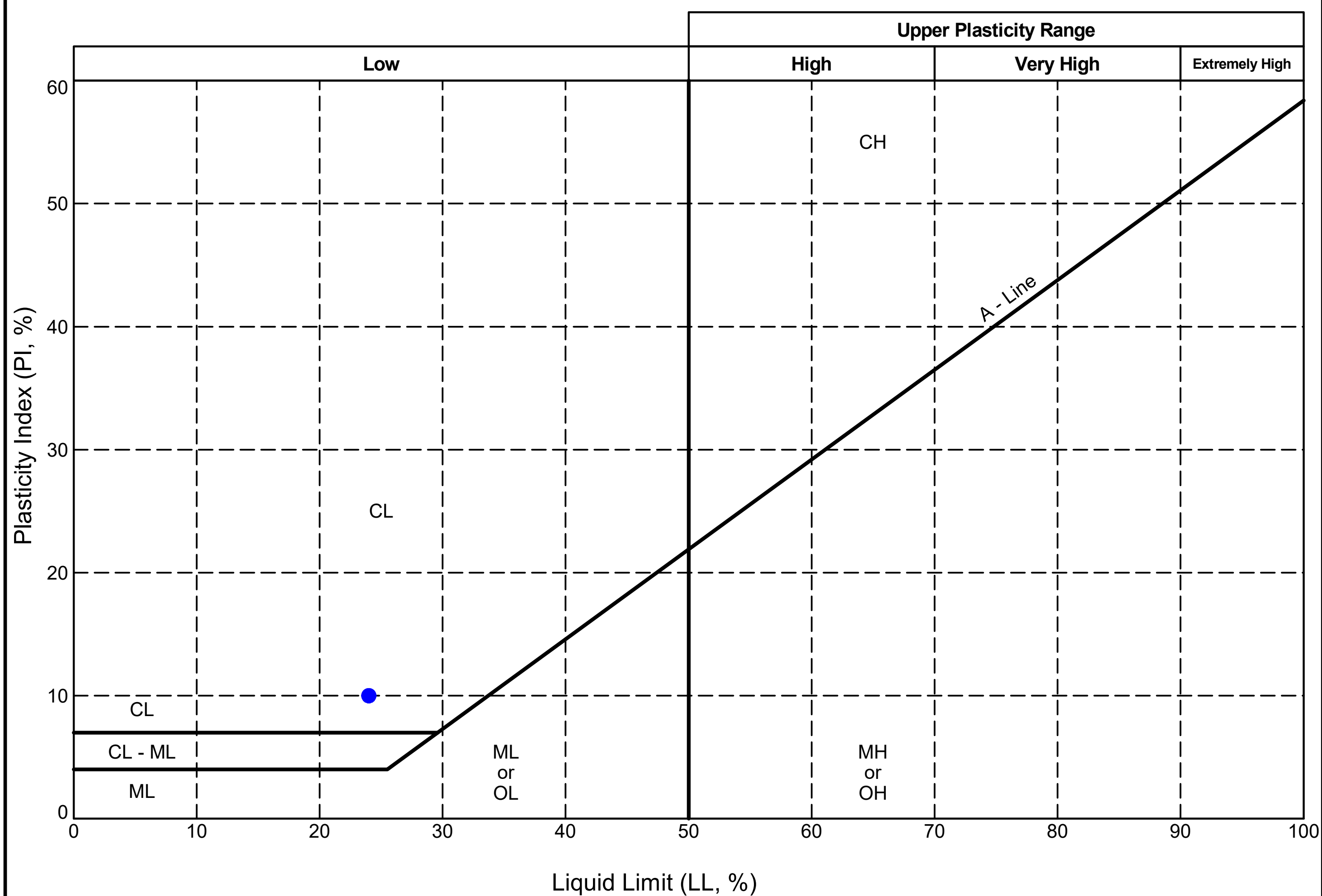
	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)	Description
●	3	SS6	4.9	73.3	24	16	8	SLIGHTLY PLASTIC

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

ATTERBERG LIMITS CHART

File No.: **1-18-0012-01**



	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)	Description
●	5	SS7	6.4	71.2	24	14	10	SLIGHTLY PLASTIC

Terraprobe
11 Indell Lane, Brampton Ontario L6T 3Y3
(905) 796-2650

Title:

ATTERBERG LIMITS CHART

File No.:

1-18-0012-01

Appendix B:

Storm and SWM Design sheets and calculations

Urbantech West, A Division of Leighton-Zec West Ltd.
2030 Bristol Circle Suite 201 Oakville, Ontario L6H 0H2
TEL: 905.829.8818
www.urbantech.com

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STORM SEWER DESIGN SHEET

10 Year Storm - Existing

55 Port Street East

City of Mississauga

PROJECT DETAILS

Project No: 17-548W
Date: 4-Dec-18
Designed by: JL
Checked by: RM

DESIGN CRITERIA

Min. Diameter =	300	mm	Rainfall Intensity =	A / (Tc+B)^c
Mannings 'n'=	0.013			
Starting Tc =	15	min	A =	1010
			B =	4.6
Factor of Safety =	10	%	c =	0.78

NOMINAL PIPE SIZE USED

STREET	FROM MH	TO MH	AREA (ha)	RUNOFF COEFFICIENT "R"	'AR'	ACCUM. 'AR'	RAINFALL INTENSITY (mm/hr)	FLOW (m3/s)	CONSTANT FLOW (m3/s)	ACCUM. CONSTANT FLOW (m3/s)	TOTAL FLOW (m3/s)	LENGTH (m)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (m3/s)	FULL FLOW VELOCITY (m/s)	INITIAL Tc (min)	TIME OF CONCENTRATION (min)	ACC. TIME OF CONCENTRATION (min)	PERCENT FULL (%)
External	EX. HW2	EX. MH46	0.11	0.90	0.10	0.10	99.2	0.027	0.016	0.016	0.043	28.4	0.56	375	0.131	1.19	15.00	0.40	15.40	33%
Port St. E	EX. MH46	EX. MH47				0.10	97.6	0.027		0.016	0.043	24.7	0.73	375	0.150	1.36	15.40	0.30	15.70	29%
Helene St. South	EX. MH47	EX. MH48				0.10	96.5	0.027		0.016	0.043	25.1	0.80	450	0.255	1.60	15.70	0.26	15.96	17%
Helene St. South	EX. MH48	EX. MH49				0.10	95.5	0.026		0.016	0.042	39.8	0.30	450	0.156	0.98	15.96	0.68	16.64	27%
	SITE	EX. MH49	0.23	0.50	0.12	0.12	99.2	0.032			0.032									
Helene St. South	EX. MH49	EX. HW1				0.21	93.1	0.055		0.016	0.071	38.0	0.25	450	0.143	0.90	16.64	0.71	17.35	50%

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TEL: 905.829.8818 FAX: 905.829.4804
www.urbantech.com

F:\Projects\17-548 (Fram and Slokker - 55 Port Street East)\Design\Storm\17-548 STM (Exisitng -10 yr):STM

PROJECT DETAILS	
Title1:	STORM SEWER DESIGN SHEET
Title2:	100 Year Storm Capture
Project Name:	55 Port Street East
Municipality:	City of Mississauga
Project No:	17-548W
Date:	4-Dec-18
Designed by:	JL
Checked by:	RM

IDF Parameters for Mississauga			
		10-yr	100-yr
$I=A/(T+b)^c$	A	1010	1450
	B	4.6	4.9
	C	0.78	0.78

CAPTURE LOCATION	AREA ID	CAPTURE POINT	Area ha	R 10 yr	R 100 yr	AR 10 yr	AR 100 yr	Flow Length m	Velocity m/s	Tc* min	I10 mm/hr	I100 mm/hr	Q10 m3/s	Q100 m3/s	Q100-Q10 m3/s	Const. flow m3/s
3 m STM Easement	1	Inlet Headwall	0.11	0.9	1.00	0.10	0.11			15.00	99.2	140.7	0.027	0.043	0.016	0.016

*Where available, Tc is calculated from design sheet or overland flow calculation

Tc calcs *where Tc = starting Tc + flow length/velocity*
(starting Tc = 15min)

Assumed Velocities for Calculation of time of Concentration
Pipe Flow Velocity= 2.0 m/s
OLF Velocity= 1.5 m/s
External Flow Velocity= 0.25 m/s

urbantech

STORM SEWER DESIGN SHEET

Proposed with 100 Year Flows

55 Port Street East

City of Mississauga

PROJECT DETAILS

Project No: 17-548W
Date: 4-Dec-18
Designed by: JL
Checked by: RM

DESIGN CRITERIA

Min. Diameter =	**300**	**mm**	**Rainfall Intensity =**	$\frac{A}{(Tc+B)^c}$
Mannings 'n'=	**0.013**			
Starting Tc =	**15**	**min**	**A =**	**1010**
			B =	**4.6**
Factor of Safety =	**10**	**%**	**c =**	**0.78**

NOMINAL PIPE SIZE USED

STREET	FROM MH	TO MH	AREA (ha)	RUNOFF COEFFICIENT "R"	'AR'	ACCUM. 'AR'	RAINFALL INTENSITY (mm/hr)	FLOW (m3/s)	CONSTANT FLOW (m3/s)	ACCUM. CONSTANT FLOW (m3/s)	TOTAL FLOW (m3/s)	LENGTH (m)	SLOPE (%)	PIPE DIAMETER (mm)	FULL FLOW CAPACITY (m3/s)	FULL FLOW VELOCITY (m/s)	INITIAL Tc (min)	TIME OF CONCENTRATION (min)	ACC. TIME OF CONCENTRATION (min)	PERCENT FULL (%)
External	HW2	EX. MH46	0.11	0.90	0.10	0.10	99.2	0.027	0.016	0.016	0.043	28.4	0.56	375	0.131	1.19	15.00	0.40	15.40	33%
Port St. E	EX. MH46	EX. MH47				0.10	97.6	0.027		0.016	0.043	24.7	0.73	375	0.150	1.36	15.40	0.30	15.70	29%
Helene St. South	EX. MH47	EX. MH48				0.10	96.5	0.027		0.016	0.043	25.1	0.80	450	0.255	1.60	15.70	0.26	15.96	17%
Helene St. South	EX. MH48	EX. MH49				0.10	95.5	0.026		0.016	0.042	39.8	0.30	450	0.156	0.98	15.96	0.68	16.64	27%
Proposed Development	Site	EX. MH49	0.23	0.67	0.15	0.15	99.2	0.042	0.033	0.033	0.075									
Helene St. South	EX. MH49	HW1				0.25	93.1	0.065		0.049	0.114	38.0	0.25	450	0.143	0.90	16.64	0.71	17.35	80%

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www.urbantech.com

F:\Projects\17-548 (Fram and Slokker - 55 Port Street East)\Design\Storm\17-548 STM (Proposed -10 yr):STM

PROJECT DETAILS

Title1:	**STORM SEWER DESIGN SHEET**
Title2:	**100 Year Storm Capture (Post Development)**
Project Name:	**55 Port Street East**
Municipality:	**City of Mississauga**
Project No:	**17-548W**
Date:	**6-Dec-18**
Designed by:	**JL**
Checked by:	**RM**

IDF Parameters for Mississauga			
		10-yr	100-yr
$I=A/(T+b)^c$	A	1010	1450
	B	4.6	4.9
	C	0.78	0.78

CAPTURE LOCATION	AREA ID	CAPTURE POINT	Area ha	R 10 yr	R 100 yr	AR 10 yr	AR 100 yr	Flow Length m	Velocity m/s	Tc* min	I10 mm/hr	I100 mm/hr	Q10 m3/s	Q100 m3/s	Q100-Q10 m3/s	Const. flow m3/s
3 m STM Easement		Inlet Headwall	0.11	0.9	1.00	0.10	0.11			15.00	99.2	140.7	0.027	0.043	0.016	0.016
Helene St.		MH 49	0.23	0.67	0.8	0.15	0.19			15.00	99.2	140.7	0.042	0.075	0.033	0.033

*Where available, Tc is calculated from design sheet or overland flow calculation

Tc calcs *where Tc = starting Tc + flow length/velocity*
(starting Tc = 10min)

Assumed Velocities for Calculation of time of Concentration
Pipe Flow Velocity= 2.0 m/s
OLF Velocity= 1.5 m/s
External Flow Velocity= 0.25 m/s

SWM CALCULATIONS
ANNUAL WATER BALANCE ANALYSIS

Project Name: 55 Port Street
Municipality: City of Missississauga
Project No.: 17-548
Date: 12-Nov-18

Prepared by: J.L.
Checked by: A.F/R.B.T.M
Last Revised: 4-Dec-18

PROPOSED SITE:

Impervious Roof	709	m^2
Green Roof	410	m^2
Soft Landscaping	420	m^2
Impervious Area (Includes Hard Landscaping)	773	m^2
TOTAL SITE AREA	**2,312**	**m^2**

AVERAGE ANNUAL PRECIPITATION: **786** **mm**

Source: http://climate.weather.gc.ca/climate_normals/results_1981_2010_e.html?searchType=stnProx&txtRadius=25&optProxType=city&selCity=43%7C35%7C79%7C37%7CMississauga&selPark=&txtCentralLatDeg=&txtCentralLatMin=0&txtCentralLatSec=0&txtCentralLongDeg=&txtCentralLongMin=0&txtCentralLongSec=0&stnID=5097&dispBack=0

INDIVIDUAL WATER BALANCE RELATIONSHIPS WITH EACH SURFACE TYPE:

Note: WWFMG Section 2.2.1.1 Figure 1a has been used to convert rainfall depths to equivalent average annual volumes.

Impervious Roof	(%)	(mm)	
Infiltration	0.0	0	
Evaporation	70.0	550	Ponding and lost through evaporation
Re-use	0.0	0	
Runoff	30.0	236	
Precipitation	100.0	786	

Green Roof	(%)	(mm)	
Infiltration	25.0	196	Soft landscaping can accept 15 mm (as infiltration or evapotranspiration) of rainfall
Evapotranspiration	25.0	196	(50% of annual volume).
Re-use	0.0	0	
Runoff	50.0	393	
Precipitation	100.0	786	

Soft Landscaping	(%)	(mm)	
Infiltration	40.0	314	Soft landscaping can accept 15 mm (as infiltration or evapotranspiration) of rainfall
Evapotranspiration	40.0	314	(83% of annual volume) without producing runoff.
Re-use	0.0	0	
Runoff	20.0	157	
Precipitation	100.0	786	

Impervious Area (Includes Hard Landscaping)	(%)	(mm)	
Infiltration	0.0	0	Zero infiltration from impervious areas
Evapotranspiration	15	118	1.5 mm of rainfall (equivalent to 15% annual volume) is lost to initial abstraction/evaporation
Re-use	0.0	0	
Runoff	85	668	
Precipitation	100.0	786	

CALCULATE OVERALL SITE-WIDE WATER BALANCE RELATIONSHIP:

The site wide water balance relationship is calculated by weighting the individual relationships (established above) based on percentage coverage of each surface type. The table below summarises the calculation.

	Impervious Roof	Green Roof	Soft Landscaping	Impervious Area (Includes Hard Landscaping)	Site-Wide	
% Land-Use Coverage	30.7%	17.7%	18.2%	33.4%	100.0%	
Infiltration (mm)	0	196	314	0	92	12%
Evapotranspiration (mm)	550	196	314	118	300	38%
Re-use	0	0	0	0	0	0%
Runoff (mm)	236	393	157	668	394	**50%**
Precipitation (mm)	786	786	786	786	786	100.0%

The analysis shows that 50.1% of rainfall leaves site as runoff on an average annual basis.

Urbantech West, A Division of Leighton-Zec West Ltd.
2030 Bristol Circle Suite 201 Oakville, Ontario L6H 0H2
TEL: 905.829.8818
www.urbantech.com

Appendix C:

Water and Wastewater Calculations (MES Engineering, November 2018)

Urbantech West, A Division of Leighton-Zec West Ltd.
2030 Bristol Circle Suite 201   Oakville, Ontario   L6H 0H2
TEL: 905.829.8818
www.urbantech.com

Municipal Engineering Solutions

November 30, 2018 Project No. 17003-20

Sent via email
Mr. Rob Merwin
Urbantech West
2030 Bristol Circle, Suite 201
Oakville, ON
L6H 0H2

Subject: 55 Port Street East Development
Water and Wastewater Calculations
City of Mississauga, Region of Peel

Dear Mr. Merwin,

Municipal Engineering Solutions ("MES") was retained by Urbantech West to calculate the water demands and sanitary flow for the proposed development located at 55 Port Street East in the City of Mississauga (Region of Peel). As part of this assignment MES was requested to calculate the flow requirements for the proposed development using Region of Peel, Fire Underwriters Survey, provincial and industry design standards to complete the Region's Single-Use Demand Table.

Development Background

The development site is located at Port Street East and Helen Street South in the City of Mississauga. The development is a 10 storey apartment building consisting of 34 residential units. The proposed water connection to the building is from the existing 300 mm diameter watermain on Port Street East within pressure Zone 1. The proposed sanitary sewer connection is to the existing 250 mm sanitary sewer which is conveyed to the existing 450 mm sanitary sewer on Port Street East.

Equivalent Population Serviced

To calculate the equivalent population for the proposed building MES used population densities for the apartment units that were provided by the Region of Peel. **Table 1** summarizes the residential population densities.

Table 1 – Equivalent Residential Population Density

Type of Development	Equivalent Population (Persons/Unit)
1 Bedroom Apartment	1.68
2+ Bedroom Apartment	2.54

Source: Region of Peel Comments

The equivalent population for the site was calculated to be 83 people. Detailed calculations are attached.

Domestic Water Usage

The domestic water demands for the development were calculated using the design criteria outlined in the Region of Peel "*Watermain Design Criteria, June 2010*". **Table 2** summarizes the average daily demand and peaking factors used for this analysis.

Table 2 - Water Design Factors

Type of Development	Average Daily Demand	Maximum Daily Demand Peaking Factor	Peak Hourly Demand Peaking Factor
Residential	280 L/capita/day	2.0	3.0

Source: Region of Peel Watermain Design Criteria, 2010

Utilizing the equivalent population and the corresponding Maximum Day and Peak Hour data from Table 2 the water demands for this development were calculated. The calculated water demands for the development are summarized in **Table 3**. Detailed calculations are attached.

Table 3 – Total Domestic Water Demands

	Average Day Demand (L/s)	Maximum Day Demand (L/s)	Peak Hour Demand (L/s)
Total Building Demands	0.27	0.54	0.81

Fire Flow Demands

The fire demands for the proposed building were calculated using the Fire Underwriters Survey ("FUS") formula outlined in the *'Water Supply For Public Fire Protection Guideline',* dated 1999. The minimum required fire flow is shown in **Table 4**. Detailed calculations are attached.

Table 4 - Fire Flow Requirements

Type of Development	Fire Flow (L/s)
Apartment Building	167

Source: Fire Underwriters Survey

As noted, the fire flow in Table 4 above was calculated using the FUS formula. **Table 5** below summarizes the criteria utilized to calculate the fire flow requirements for the apartment building as well as the assumptions made. Once the detailed design data (specifics) for this building are finalized the assumptions noted in Table 5 and in the FUS calculation must be reviewed and confirmed by the appropriate designer (architect or sprinkler system designer) and any design/criteria changes required are to be reported to MES.

Table 5 – FUS Criteria/Assumptions

	Type of Development
	Mid-Rise Residential
Type of Construction	Non-Combustible Construction (Must Comply With FUS Fire Resistive Rating) (Steel Frame, Concrete Floor Slabs)
Occupancy Type	Limited Combustible
Fire Protection (Sprinkler/Firewalls)	Fully Supervised Sprinkler System
Area Considered	Total building area was assumed to be 6316.0 m^2 (as provided by Urbantech West)

Note: For Additional Information on FUS Criteria Refer to Water Supply for Public Protection Guide, Fire Underwriters Survey, 1999

Hydrant Test

A hydrant test was performed on Port Street East on May 8, 2018 by Jackson Waterworks. The results of the hydrant test are attached.

Sanitary Sewer Flow

The sanitary flow for the development was calculated using the design criteria outlined in the Region of Peel "*Sanitary Sewer Design Criteria, July 2009*". **Table 6** summarizes the sanitary flow and infiltration allowance used for this analysis.

Table 6 - Sanitary Design Factors

Type of Development	Sewage Flow
Domestic Sewage Flow (Population <1000 people)	0.013 m^3/sec
Infiltration	0.0002 m^3/sec/Ha

Utilizing the equivalent population and the corresponding rates from Table 6 the sanitary flow for this development was calculated. The calculated sanitary flow for the development is summarized in **Table 7**. Detailed calculations are attached.

Table 7 – Total Sanitary Flow

	Sanitary Flow (L/s)
Total Sanitary Sewer Effluent	13.05

We trust you find this report satisfactory. Should you have any questions or require further clarification, please call.

Yours truly,

Municipal Engineering Solutions

Kristin St-Jean, P.Eng.

/KS

File Location: C:\Users\Acer\Documents\Projects\17003-20 Port Street, Mississauga\5.0 Report\17003-20 Port St Calculations_20181130.docx

Attachments:

Connection Single Use Demand Table
Region of Peel Design Criteria
Region of Peel Comments
Domestic Water Usage Calculations
Fire Underwriters Survey (FUS) Calculations
Hydrant Test Results
Sanitary Sewer Flow Calculations

Connection Single Use Demand Table

WATER CONNECTION

Connection point [3]			
Existing 300 mm diameter watermain on Port Street East			
Pressure zone of connection point		Zone 1	
Total equivalent population to be serviced [1]		83 People	
Total lands to be serviced		0.23 Hectares	
Hydrant flow test			
Hydrant flow test location			
55 Port Street East			
	Pressure (kPa)	Flow (in l/s)	Time
Minimum water pressure	579.2	158.7	10:00 am
Maximum water pressure	634.3	0	10:00 am

No.	**Water demands**		
	Demand type	**Demand**	**Units**
1	Average day flow	0.27	l/s
2	Maximum day flow	0.54	l/s
3	Peak hour flow	0.81	l/s
4	Fire flow [2]	167	l/s
Analysis			
5	Maximum day plus fire flow	167.54	l/s

LICENSED PROFESSIONAL ENGINEER
K. E. ST-JEAN
100161507
Nov 30/2018
PROVINCE OF ONTARIO

WASTEWATER CONNECTION

Connection point [4] Existing 250 mm diameter sewer at 55 Port Street East	
Total equivalent population to be serviced [1]	83 People
Total lands to be serviced	0.23 Ha
6 Wastewater sewer effluent (in l/s)	13.05 L/s

[1] The calculations should be based on the development estimated population (employment or residential).

[2] Please reference the Fire Underwriters Survey Document

[3] Please specify the connection point ID

[4] Please specify the connection point (wastewater line or manhole ID)
Also, the "total equivalent popopulation to be serviced" and the "total lands to be serviced" should reference the connection point. (The FSR should contain one copy of Site Servicing Plan)

Please include the graphs associated with the hydrant flow test information table
Please provide Professional Engineer's signature and stamp on the demand table
All required calculations must be submitted with the demand table submission.

Region of Peel Design Criteria

Equivalent Population by Unit

Type of Development	**Equivalent Population Density**
	(Person/Unit)
Apartment - 1 Bedroom	1.68
Apartment - 2+ Bedroom	2.54

Source: Population per Unit provided by the Region of Peel

Water Design Factors

Residential	
Average Daily Demand (L/person/day)	280
Maximum Day Factor	2.0
Peak Hour Factor	3.0

Source: Region of Peel Watermain Design Criteria, June 2010

Sanitary Design Factors

Design Flow by Population	**Sewage Flow (m^3/sec)**
Domestic Sewage Flow (<1000 persons)	0.013
Infiltration by Hectare (m^3/sec/Ha)	0.0002

Source: Region of Peel Sanitary Sewer Design Criteria, July 2009

Region of Peel	INFO REPORT	The consultant is required to complete and submit the Single-Use Demand table for the Region to fulfill our modelling requirements and determine the proposal?s impact to the existing system. The demand table should be in digital format and accompanied by the supporting graphs for the hydrant flow tests and shall be stamped and signed by the Professional Consulting Engineer. This demand table will be required prior to RZ/OZ Approval. For the design flow calculations, please use the following PPU?s: Apartment (2+ bedrooms) ? 2.54 Apartment (1 bedroom) ? 1.68

RESIDENTIAL

Population

Unit Type	No. of Units	PPU	Population
1 Bedroom Apartment	4	1.68	6.7
2+ Bedroom Apartment	30	2.54	76.2
Total Population			**83**

Water Demands

Demand Type	Population	Demand Rate
Average Day	83	280 L/capita/day
Average Day Water Demand		23240 L/day
		0.27 L/s

Total Water Demands

Demand Type	Peaking Factor	Water Demands
Average Day		0.27 L/s
Maximum Day	2.0	0.54 L/s
Peak Hour	3.0	0.81 L/s

TOTAL

Population

Total Population	83

Total Water Demands

Demand Type	**Demand (L/s)**
Average Day	0.27
Maximum Day	0.54
Peak Hour	0.81

Calculations are based on "Water Supply for Public Fire Protection Guide" by Fire Underwriters Survey

Municipal Engineering Solutions

FUS CALCULATION

Project:	55 Port Street East	**Building Type/Block #**	Residential Mid Rise
Project Number:	17003-20	**Firewalls/Sprinkler:**	Sprinklered
Project Location:	City of Mississauga, Peel Region	**Number of Units/Unit #'s**	10 Storey - 34 units

1.0 FUS Formula

$$F = 220C\sqrt{A}$$

where: F = required fire flow in litres per minute;
C = the Coefficient related to the type of construction; and
A = the total floor area in square metres (including all storeys but excluding basements at least 50% below grade)[a]

NBC Occupancy	Group C	
Type of Construction[b]	non-combustible construction	
Storeys	10	
C =	0.8	
A =	6316.0	Gross Floor Area (excludes underground parking)
F =	**14000**	**L/min**

2.0 Occupancy Adjustment

Type of Occupancy[c]	limited combustible	
Hazard Allowance	-0.15	
	-2100	L/min
Adjusted Fire Flow	**11900**	**L/min**

3.0 Sprinkler Adjustment

		Credit	Total
NFPA 13 sprinkler standard	YES	30%	50%
Standard Water Supply	YES	10%	
Fully Supervised system	YES	10%	

Sprinkler Credit **5950 L/min**

4.0 Exposure Adjustment

		Percent	Total*
North Side			30%
Distance to Building (m)	20.1 to 30	10%	
Length (ft) by height in storeys	over 120		
South Side			
Distance to Building (m)	over 45	0%	
Length (ft) by height in storeys	over 120		
East Side			
Distance to Building (m)	3.1 to 10	20%	
Length (ft) by height in storeys	over 120		
West Side			
Distance to Building (m)	over 45	0%	
Length (ft) by height in storeys	over 120		

*max 75%

Exposures Surcharge **3570 L/min**

Total Required Fire Flow	**10000**	**L/min**
(rounded)	**167**	**L/sec**

a) For fire-resistive buildings, consider the two largest adjoining floors plus 50% of each of any floors immediately above them up to 8, when vertical openings are inadequately protected. If the vertical openings and and exterior vertical communications are properly protected, consider only the area of the largest floor plus 25% of each of the two immediately adjoining floors
b) Wood frame=1.5, Ordinary=1.0, Non-combustible=0.8, Fire-resistive=0.6
c) Non-combustible=-25%, Limited combusitble=-15%, Combustible=0, Free burning=+15%, Rapid burning=+25%

55 Gilbank Drive, Aurora, Ontario L4G 6H9
Tel: 905.726.1016 Cell: 416.434.0186 Fax: 905.726.1225

JACKSON WATERWORKS

Telephone: (905) 547-6770
Toll Free: (800)-734-5732
E-mail: jww@bellnet.ca
Website: www.jacksonwaterworks.ca

Mr. Connor Wright
Urbantech West
2030 Bristol Circle, Suite 201
Oakville Ontario **L6H 0H2**

09 May 2018

Jackson Waterworks has recently completed fire hydrant flow testing at 55 Port Street East in Mississauga.

We define the Test Hydrant as the one being flowed, and the Base Hydrant as the one where static and residual pressures are recorded. Wherever possible, we inspect the secondary valve for the Test Hydrant to make sure it is in the fully open position. Likewise, we count the number of turns needed to open the Test Hydrant (to make sure it is opening completely).

The secondary valve for the Test Hydrant could not be located for inspection.

Testing was completed in accordance with NFPA 291 guidelines.

There were no irregularities to report.

Trusting this meets with your approval, we are...

Yours truly,

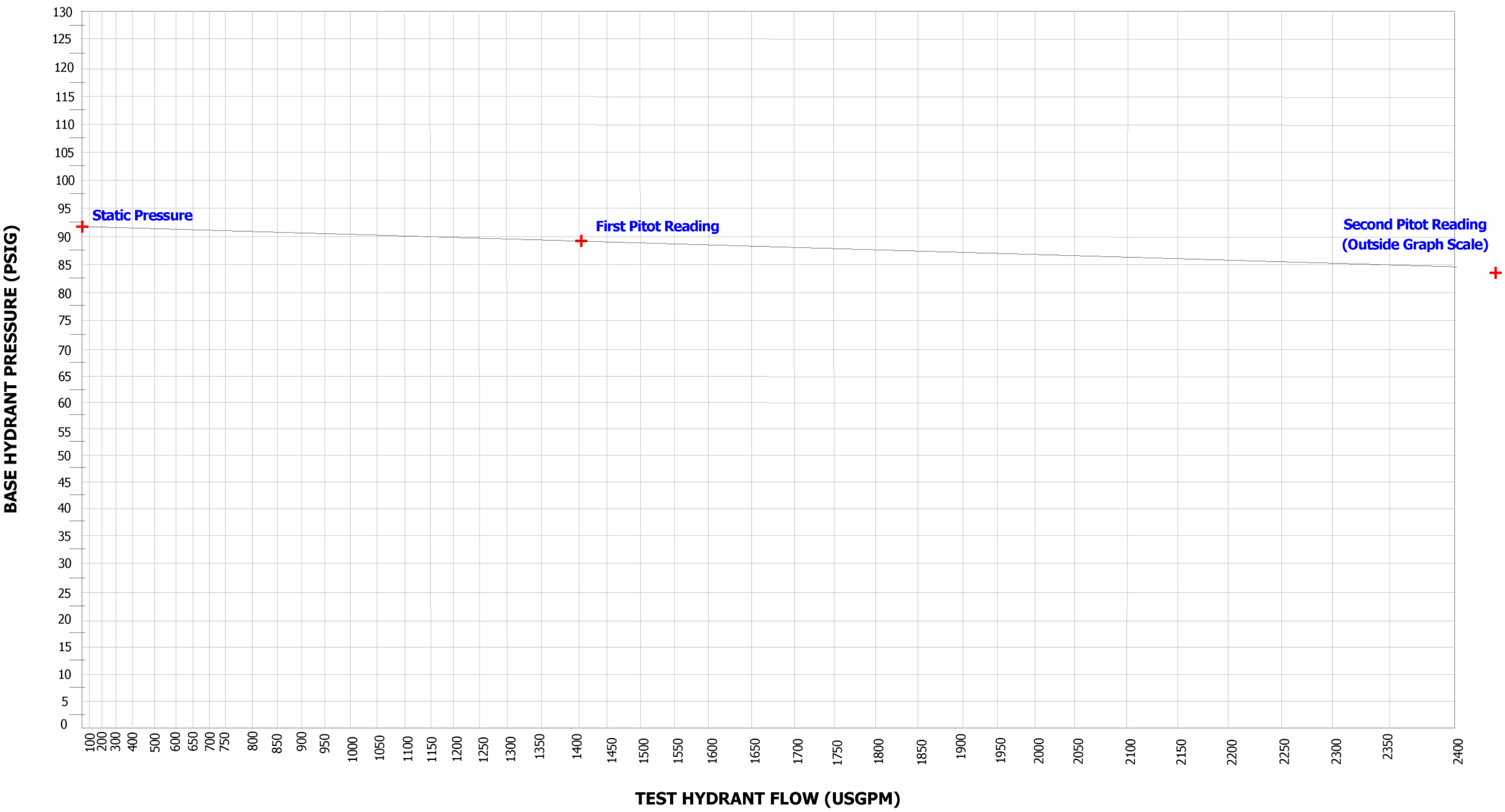
Mark Schmidt
Jackson Waterworks

221 Sherman Avenue North, Hamilton, Ontario L8L 6N2

JACKSON WATERWORKS
Telephone: (905) 547-6770
Toll Free: (800)-734-5732
E-mail: jww@bellnet.ca
Website: www.jacksonwaterworks.ca

FIRE HYDRANT FLOW TEST RESULTS

TEST #1 of 1



No. of Ports Open	Port Dia. (in)	Pitot Reading (psig)	Pitot Conversion (usgpm) Conversion Factor = 0	Residual Pressure (psig)
1	2.50	70	1404	89
2	2.50	56/56	2512	84
THEORETICAL FLOW @ 20psi			**7810**	

Test Date	08 May 2018
Test Time	10:00am
Pipe Diameter (in)	12
Static Pressure (psig)	92

Site Information		
Site Name or Developer Name	Urbantech West	Engineer: Not Provided
Site Address/Municipality	55 Port Street East, Mississauga	
Location of Test Hydrant	In Front of 55 Port Street East	
Location of Base Hydrant	Port Street East, 1st West of Hurontario Street	
Comments	Testing has been completed in accordance with NFPA-291 guidelines wherever and whenever possible and practical. Conversion factors for pitot tube readings have been used depending on hose nozzle internal design and installation profile. Refer to attached cover letter for additional information.	
Verified By	Mark Schmidt	

RESIDENTIAL

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Design Flow

Demand Type	Population	Demand Rate
Domestic Flow	83	0.013 m^3/sec
Domestic Sanitary Sewage Flow		13.0 L/s

Infiltration

Demand Type	Area (sq. m)	Demand Rate
Infiltration	2300	0.0002 m^3/sec/Ha
Domestic Sanitary Sewage Flow		0.05 L/s

Total Sanitary Flow

Demand Type		Sanitary Flow
Domestic and Infiltration		13.05 L/s

TOTAL

Population

Total Population	83

Total Sanitary Flow

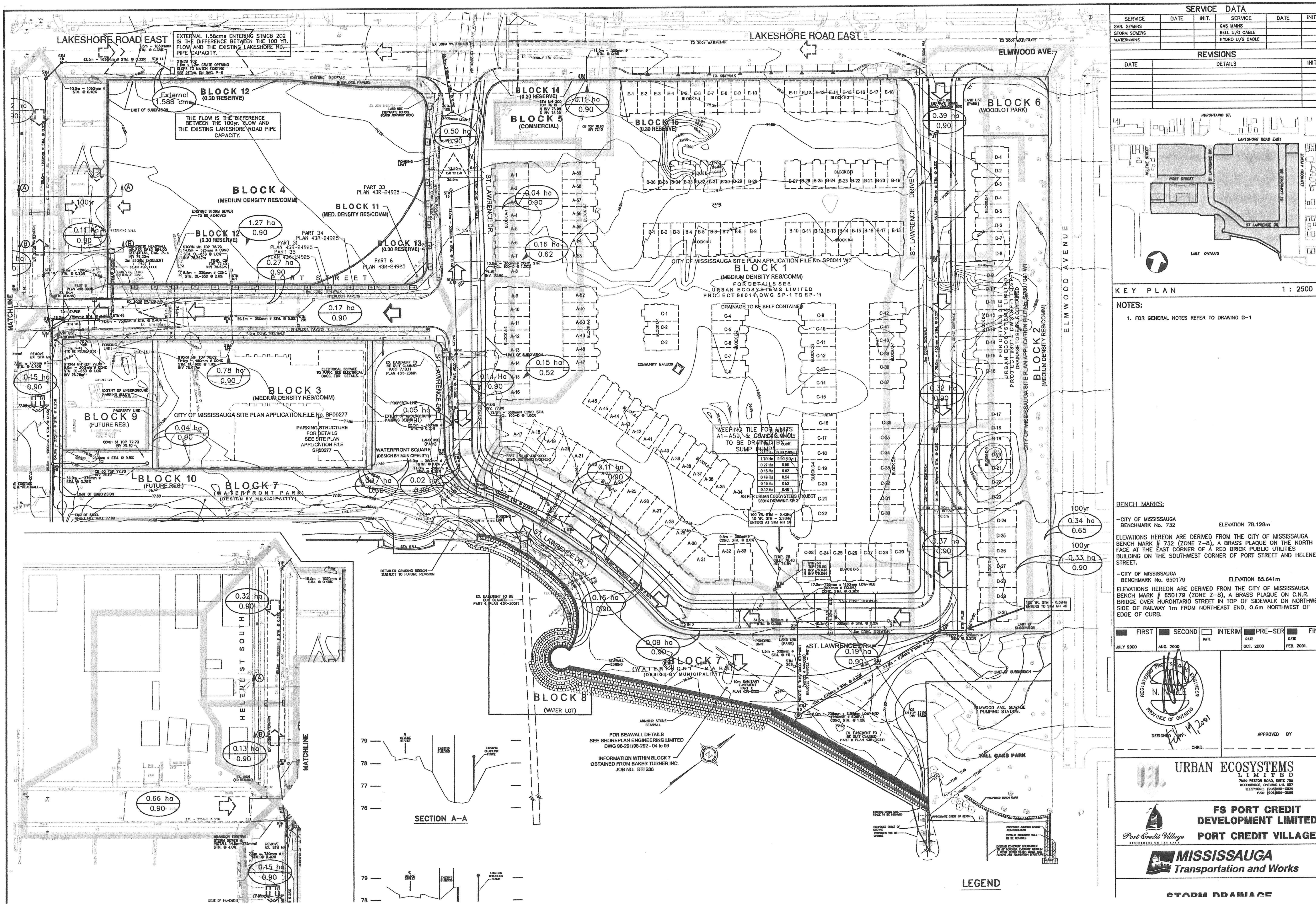
Demand Type	Demand (L/s)
Domestic Flow	13.05

Drawings

Drawing G-4 Storm Drainage Area Plan (Urban Ecosystems, February 2001)
Drawing GSP-1 Site Grading Plan and Site Servicing Plan

Urbantech West, A Division of Leighton-Zec West Ltd.
2030 Bristol Circle Suite 201 Oakville, Ontario L6H 0H2
TEL: 905.829.8818
www.urbantech.com

DRAWINGS G-4 STORM DRAINAGE PLAN (URBAN ECOSYSTEMS)



File: F:\PROJECTS\17-548 (FRAM AND SLOKKER - 55 PORT STREET EAST)\DRAWINGS\00_FUNCTIONAL\02_FD_PRODUCTION DWGS\GSP-1 GRADING AND SERVICING PLAN.DWG

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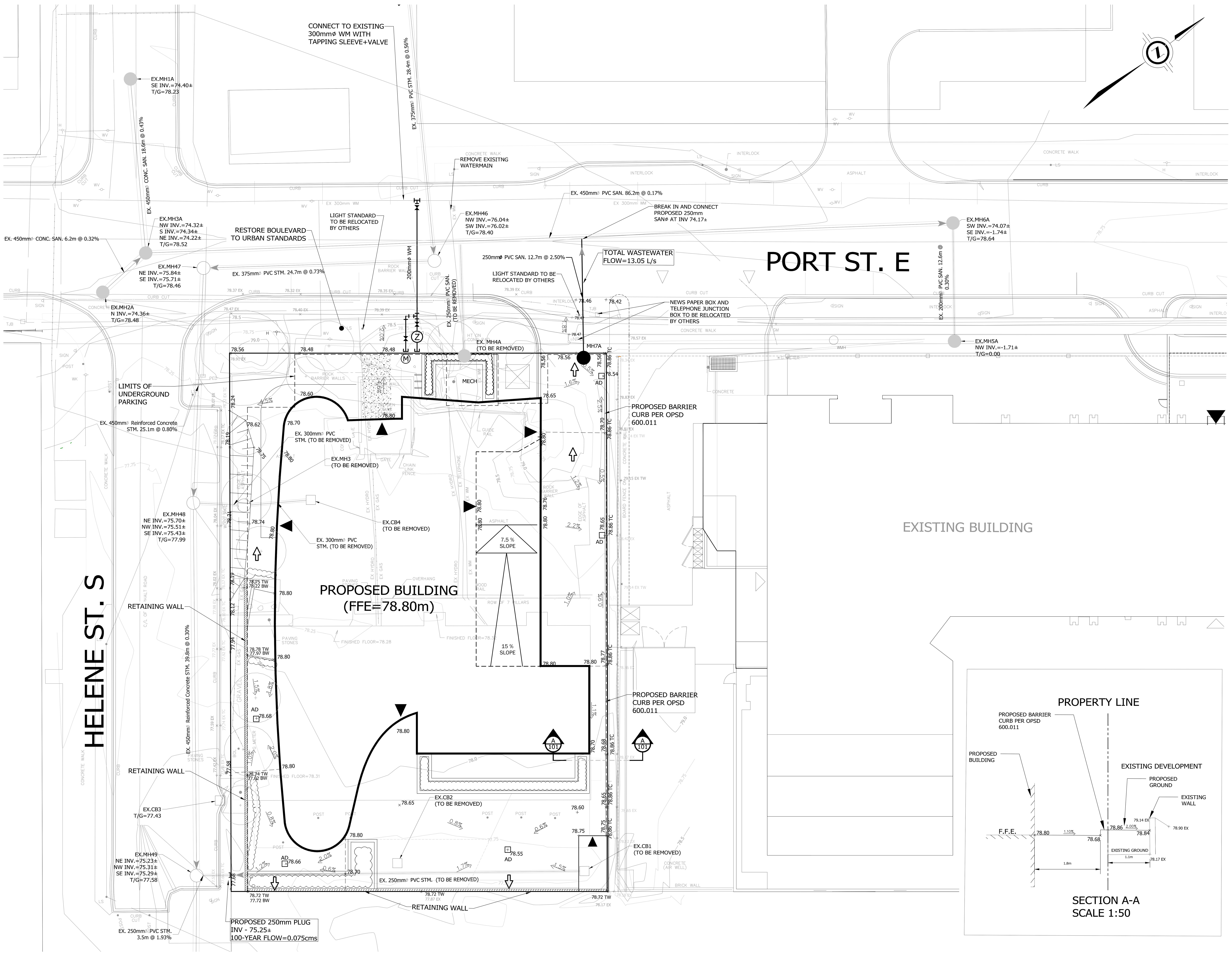

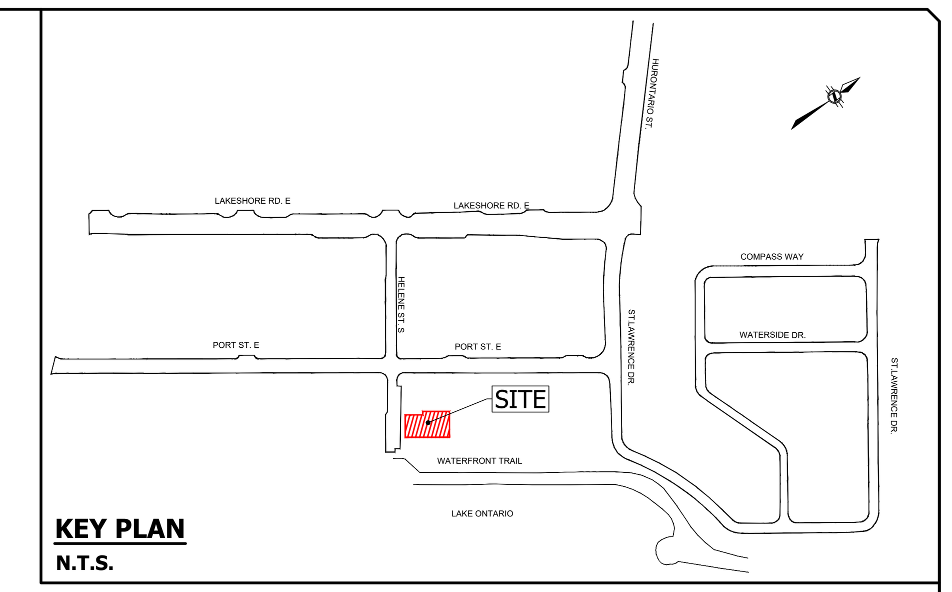

KEY PLAN
N.T.S.

LEGEND

	SITE BOUNDARY		SINGLE/REARLOT CATCHBASIN
	BUILDING OUTLINE		DOUBLE CATCHBASIN
162.35	PROPOSED GRADE		EXISITING SAN MANHOLE
	EXISTING GRADE		EXISITING STM MANHOLE
	EXISTING MAJOR CONTOUR		TREE
	EXISTING MINOR CONTOUR		
	WALKWAY		
	DRAINAGE SPLIT		MAXIMUM 3:1 SLOPE (UNLESS SPECIFIED)
1.00%	PROPOSED ROAD GRADE AND FLOW DIRECTION		
	AREA DRAIN		
	EMERGENCY FLOW DIRECTION		

LOCAL BENCHMARK

TOP NUT OF FIRE HYDRANT LOCATED ALONG THE SOUTH SIDE OF PORT STREET, APPROXIMATELY 1.2m WEST OF THE ENTRANCE TO No. 55 PORT STREET. ELEVATION=79.58m

ELEVATION NOTE

ELEVATIONS SHOWN ON THIS PLAN ARE RELATED TO GEODETIC DATUM AND ARE DERIVED FROM THE CITY OF MISSISSAUGA BENCH MARKS:

No. 731 ELEVATION=81.58m
No. 732 ELEVATION=78.128m
No. 3470 ELEVATION=78.304m

PROPERTY LINE
PROPOSED BARRIER CURB PER OPSD 600.011
PROPOSED BUILDING
EXISTING DEVELOPMENT
PROPOSED GROUND
EXISTING WALL
F.F.E.
EXISTING GROUND

SECTION A-A
SCALE 1:50

No.	REVISION	DATE	
2	2nd SUBMISSION - OFFICIAL PLAN AND ZONING BY-LAW AMMENDMENT	DEC 7, 2018	R.B.T.M.
1	1st SUBMISSION - OFFICIAL PLAN AND ZONING BY-LAW AMMENDMENT	FEB 22, 2018	R.B.T.M.

SUBMISSIONS: FIRST: SECOND: INTERIM: PRE-SERVICING: FINAL:

FRAM + Slokker
55 PORT STREET EAST

CITY FILE No. OZ/OPA 18 7 REGION FILE No. TBD

MISSISSAUGA Transportation and Works

Region of Peel Working for you

urbantech west

Urbantech West, A Division of Leighton-Zec West Ltd.
2030 Bristol Circle, Suite 201 Oakville, Ontario L6H 0H2
tel: 905.829.8818 fax: 905.829.4804
www.urbantech.com

REGISTERED PROFESSIONAL ENGINEER R.B.T. MERWIN 100009772 DEC 7, 2018 PROVINCE OF ONTARIO

GRADING AND SERVICING PLAN

DESIGNED:	J.L	CHECKED:	R.B.T.M.	PROJECT No.:	17-548W
DRAWN:	J.L	DATE:	NOVEMBER 2017	SHEET No.:	
SCALE:	1:200			DRAWING No.:	GSP-1