Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

GEOTECHNICAL INVESTIGATION
WESTMINSTER PLACE SENIOR RESIDENCE
4150 WESTMINSTER PLACE
MISSISSAUGA, ONTARIO

Prepared for: St. Luke's Dixie Senior Residence Corp.
4150 Westminster Place
Mississauga, Ontario
L4W 3Z7

Attention: Mr. Dave Estabrook

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Distribution

1 Electronic Copy	-	St. Luke's Dixie Senior Residence Corp.
1 Copy	-	Terraprobe Inc., Brampton

Terraprobe Inc.

Greater Toronto
11 Indell Lane
Brampton, Ontario L6T 3Y3
(905) 796-2650 Fax: 796-2250
brampton@terraprobe.ca

Hamilton – Niagara
903 Barton Street, Unit 22
Stoney Creek, Ontario L8E 5P5
(905) 643-7560 Fax: 643-7559
stoneycreek@terraprobe.ca

Central Ontario
220 Bayview Drive, Unit 25
Barrie, Ontario L4N 4Y8
(705) 739-8355 Fax: 739-8369
barrie@terraprobe.ca

Northern Ontario
1012 Kelly Lake Rd., Unit 1
Sudbury, Ontario P3E 5P4
(705) 670-0460 Fax: 670-0558
sudbury@terraprobe.ca

www.terraprobe.ca

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1 INTRODUCTION

Terraprobe Inc. (Terraprobe) was retained by St. Luke’s Dixie Senior Residence Corp. to conduct a geotechnical investigation for a proposed infill development located at 4150 Westminster Place, in the City of Mississauga.

This report encompasses the results of the geotechnical investigation conducted for the proposed development to determine the prevailing subsurface soil, bedrock and ground water conditions, and provide geotechnical engineering design recommendations for the proposed building foundations, earth pressure and seismic design parameters, basement floor slab and drainage, and shoring considerations. Geotechnical comments are also included on pertinent construction aspects, excavation, backfill and ground water control.

Terraprobe was also retained to conduct an Hydrogeologic Study and Phase One Environmental Site Assessments for the subject site. The results of the investigations will be submitted under the separate covers.

2 SITE AND PROJECT DESCRIPTION

The site is located in the northwest quadrant of the intersection of Westminster Place and Rathburn Road East, in the City of Mississauga, with a municipal address of 4150 Westminster Place, Mississauga. The general location of the site is presented on Figure 1.

The project site is currently occupied with a 3-storey apartment building block and at-grade asphalt parking lot as well as landscaped areas. The asphalt parking lot is located in the southern portion of the site. It is proposed to demolish the existing common room in the parking lot area to facilitate redevelopment of the site to include an eight-storey building resting on a one-level underground parking structure (P1).

Terraprobe was provided with the following design drawings for review to prepare this report,

- *Westminster St. Luke’s Seniors Affordable Housing*, prepared by Kearns Mancini Architects, dated April 4, 2022.

The design drawing indicated that the P1 finished floor elevation (FFE) will be set at Elev. 137.4 m

3 INVESTIGATION PROCEDURE

The field investigation was conducted on July 31, November 3 and 4, 2020, and July 27, 2022, consisted of drilling, coring and sampling a total of six (6) boreholes, extending to depths varying from about 3.2 m (Borehole 6) to 8 m (Boreholes 1 and 4) below grade. Two (2) boreholes (Boreholes 2 and 5) were augured to 4.3 m and 5.2 m depths below grade, respectively to install monitoring wells for the hydrogeological assessment. The approximate locations of the boreholes are shown on the enclosed Borehole Location Plan (Figure 2).

The boreholes were drilled by a specialist drilling contractor using a truck mounted drill rig power auger. The borings were advanced using continuous flight solid stem augers and were sampled at 0.75 m interval with a conventional 50 mm diameter split barrel samplers when the Standard Penetration Test (SPT) was carried out (ASTM D1586). Bedrock coring (using HQ bit size) was carried in two (2) selected boreholes (Boreholes 1 and 4) to confirm and characterize bedrock. The field work (drilling, coring, sampling and testing) was observed and recorded by a member of our field engineering staff, who logged the borings and examined the soil and rock samples as they were obtained.

All samples obtained during the field 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 two (2) selected native soil samples (Borehole 1, Sample 2; and Borehole 4, Sample 3). The laboratory test results are plotted on the enclosed Borehole Logs at respective sampling depths. The results of Sieve and Hydrometer analysis are also summarized in Section 4.2 of this report, and appended.

Bedrock core samples were retrieved and stored in wooden boxes and transported to our laboratory for further visual examination by a Geotechnical Engineer. The rock core logs and photographs are appended. Compressive strength test was performed on two (2) selected cores at varying depths from Boreholes 1 and 4 in accordance with MTO LS-410 (CSA-A23.2-14C). The geotechnical laboratory testing results are plotted on the Core Logs at respective sampling depths, and presented in Appendix B.

Water levels were measured in boreholes upon completion of drilling. Monitoring wells comprising 50 mm diameter PVC pipes were installed in selected five (5) boreholes (Boreholes 3, 4 and 6 in 2020 and Boreholes 2 and 5 in 2022) to facilitate ground water monitoring and 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 August 28, 2020, August 3, 2022, September 6, 2022, and October 13, 2022. The results of ground water monitoring are presented in Section 4.3 of this report and plotted on the enclosed Borehole Logs.

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 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 Stratigraphy

In summary, the boreholes encountered pavement structure at the ground surface underlain by earth fill zone, extending to depths varying from about 0.6 to 0.8 m below grade, which was underlain by clayey silt till deposit, extending to 3.0 m depth below grade. The till deposit graded into the bedrock of Georgian Bay Formation.

4.1.1 Pavement Structure

An asphalt pavement structure, consisting of 80 mm thick asphaltic concrete underlain by 80 to 150 mm thick granular base course was encountered in Boreholes 1 to 5 at the ground surface. A 65 mm thick concrete paver pavement structure was encountered in Borehole 6 at the ground surface.

4.1.2 Earth Fill

A zone of earth fill materials was encountered in Boreholes 1 to 5 beneath the pavement structure and extended to depths varying from about 0.6 m (Boreholes 1, 2 and 5) to 0.8 m (Boreholes 3 and 4) below existing grade. The earth fill materials consisted of mixed composition comprising clayey silt with some sand and trace amounts of gravel. Sporadic organic presence was noted within the fill materials at varying depths. A sand earth fill zone was encountered beneath the concrete pavers in Borehole 6 and extended to about 0.8 m depth below grade.

Standard Penetration Test results (N-values) obtained from the clayey silt earth fill zone varied from 7 to 18 blows per 300 mm of penetration, indicating a firm to very stiff consistency. The N-value obtained from the sand fill zone was 6 blows per 300 mm of penetration, indicating a loose relative density. The in-situ moisture contents of the earth fill samples ranged from 6 to 13 percent by mass, indicating a moist condition.

4.1.3 Glacial Till

Glacial till deposit was encountered in all boreholes beneath the earth fill zone at depths varying from about 0.6 m (Boreholes 1, 2 and 5) to 0.8 m (Boreholes 3, 4 and 6) blow grade and extended to about 3 m depth below grade. The till deposit predominately consisted of clayey silt with varying amounts of sand (some sand to sandy) and trace to some gravel. Shale fragments were also encountered at the bottom till deposit.

N-values obtained from the undisturbed till deposit ranged from 15 to 60 blows per 300 mm of penetration and 50 blows per 100 to 150 mm of penetration, indicating a very stiff to hard consistency. The in-situ moisture contents of the glacial till samples ranged from 4 to 13 percent by mass, indicating a moist condition.

4.1.4 Shale

The glacial till deposits graded into till-shale complex/weathered shale (Bedrock of Georgian Bay Formation) in each borehole at about 3.0 m depth below grade. Rock coring was carried out in Boreholes 1 and 4, extending to about 8.0 m depth below grade. Borehole Logs and Rock Core Logs are provided in Appendix A. Photographs of the recovered rock core samples are provided in Appendix C. Based on the results of the visual inspection of the recovered rock cores and field observations, the bedrock depth and elevations are tabulated as follows:

Borehole	Ground Surface Elevation	Partially Weathered (Zone II) Bedrock		Unweathered/Sound (Zone I) Bedrock		Method of Observation
		Depth	Elevation	Depth	Elevation	
Borehole 1	139.5 m	3.0 m	136.5 m	4.7	134.8 m	Confirmed with Rock coring
Borehole 2	139.6 m	3.0 m	136.6 m	N/A	N/A	Drilling observations, SPT samples
Borehole 3	139.9 m	3.0 m	136.9 m	N/A	N/A	Drilling observations, SPT samples
Borehole 4	139.8 m	3.0 m	136.8 m	4.9	134.9 m	Confirmed with Rock coring
Borehole 5	139.6 m	3.0 m	136.6 m	N/A	N/A	Drilling observations, SPT samples
Borehole 6	139.8 m	3.0 m	136.8 m	N/A	N/A	Drilling observations, SPT samples

The bedrock beneath the site consists of the Georgian Bay Formation, which a deposit predominantly comprises thin to medium bedded grey shale of Ordovician age. The shale contains interbedded grey calcareous shale, limestone/dolostone and calcareous sandstone (conventionally grouped together as "limestone") which are discontinuous and nominally 25 to 125 mm thick. The strength of the bedrock ranged from weak to strong based on the field estimate method. Compressive strength test was performed on two (2) selected cores in accordance with MTO LS-410 (CSA-A23.2-14C), which further verified the strength classification. The result of the compressive strength test and unit weight was tabulated below,

Borehole	Depth (m)	Unit Weight (kN/m^3)	Compressive Strength (MPa)
Borehole 1	6.1 - 6.3	25.0	16.8
Borehole 4	7.2 - 7.4	25.5	52.7

There is typically a zone of weathering at the contact between the rock of the Georgian Bay Formation and the glacial soil overburden. In the Ontario Ministry of Transportation and Communications document RR229, there is reproduced from Skempton, Davis and Chandler, a typical weathering profile of 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

The surface of the rock having been scoured by the base of glacial ice, Shale Zone III and IV are not usually present in an identifiable form. At the base of the glacial till deposit, there is sometimes found a zone of silt and fragmented shale that can be interpreted as the lowest portion of the till or as partially weathered Zone III rock. The distinction is subjective and depends on the investigator. The differences between the partially weathered classes of rock are not profound.

Rock Quality Designation (RQD) refers to the total length of those pieces of sound core which are 100 mm or greater in length in a core run, expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural fractures or bedding, and not machine induced or subsequent artificial breaks. The RQD of the recovered rock cores varied between 0 to 92 percent. The details of RQD are summarized in the following table,

Borehole	Rock Coring Run No.	RQD	Rock Mass Quality	Depth/Elevation Top of Sound Bedrock
Borehole 1	R1	0	Very Poor	4.7 m depth below grade Elev. 134.8 m
	R2	56	Fair	
	R3	64	Fair	
	R4	83	Good	
	R5	82	Good	
Borehole 4	R1	0	Very Poor	4.9 m depth below grade Elev. 134.9 m
	R2	20	Very Poor	
	R3	77	Good	
	R4	92	Excellent	

4.2 Geotechnical Laboratory Test Results

The geotechnical laboratory testing consisted of natural water content determination on all samples, while Sieve and Hydrometer analysis test was conducted on selected native 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	
Borehole1, Sample 2	1.0	4	27	52	17	SANDY SILT TILL some clay, trace gravel
Borehole 4, Sample 3	1.8	15	13	52	20	CLAYEY SILT TILL some sand, some gravel

4.3 Ground Water

Observations pertaining to the depth of water level and caving were made in the boreholes immediately after completion of drilling and are noted on the enclosed Borehole Logs. Monitoring wells were installed in Boreholes 2, 3, 4, 5 and 6 to facilitate ground water level monitoring. Water levels in the monitoring wells were measured on August 28, 2020, August 3, 2022, September 6, 2022, and October 13, 2022, and are noted on the enclosed Borehole Logs and appended. A summary of the borehole details and water level reading is provided as follows:

Borehole No.	Depth of Boring below Grade (m)	Water Level Depth/Elevation (m) in Monitoring wells on			
		Aug 28, 2020	Aug 3, 2022	Sept 6, 2022	Oct 13, 2022
Borehole 2	4.3	n/a	2.1/137.5	2.4/137.2	2.4/137.2
Borehole 3	3.3	2.6 /137.3	2.9/137.0	2.6/137.3	2.7/137.2
Borehole 4	7.9	1.1 /138.7	1.2/138.6	1.5/138.3	1.6/138.2
Borehole 5	5.2	n/a	3.3/136.3	3.3/136.3	3.5/136.1
Borehole 6	3.2	2.3 /137.5	2.2/137.6	2.2/137.6	2.3 /137.5

Construction dewatering at adjacent sites, existing building drains or dewatering systems, and seasonal fluctuations may cause significant changes to the depth of the groundwater table over time. Additional information pertaining to groundwater at the site is discussed in the hydrogeological report by Terraprobe under a separate cover (File No. 1-20-0258-46).

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 Foundations

The proposed infill development would include an eight-storey building resting on a one-level underground parking structure. The P1 FFE will be set at Elev. 137.4 m while the ground floor FFE is set at Elev. 104.15 m.

Six (6) boreholes were advanced within or in the vicinity of the proposed building footprint. The boreholes encountered the pavement structure at the ground surface underlain by the earth fill zone extending to depths varying from about 0.6 to 0.8 m below grade, which was in turn underlain by the glacial till deposit, extending to about 3 m depth below grade. The till deposit graded into shale bedrock at Elev. 136.5 to 136.9 m (about 3.0 m depth below grade). The sound/unweathered bedrock may be at Elev. 134.8 m± (about 4.7 to 4.9 m depth below grade).

The underside of the spread footings may be at Elev. 136.0 m± (1.4 m depth allowance for the footing depth and frost protection) while the top of bedrock may be encountered at Elev. 136.5 to 136.9 m. Therefore, foundation subgrade is expected to consist of the partially weathered (Zone II) shale bedrock. A maximum factored geotechnical resistance at ULS of 6,000 kPa and a maximum net geotechnical reaction at SLS of 3,000 kPa may be used for foundations designed on weathered (Zone II) bedrock. A minimum foundation embedment of 300 mm into the weathered bedrock must be provided.

Footings stepped from one level to another supported on the bedrock should be designed at a slope not exceeding 1 vertical to 1 horizontal in conjunction with the above bearing pressures. There must be a minimum of 500 mm separation between the edge of any footing and the top of a sloped/vertical rock cut down to another footing.

The foundation subgrade must be inspected and approved by Terraprobe to confirm that it consists of bedrock as intended/specified by the project design. The depth of extra excavation required for the foundations to be supported on sound bedrock will be determined in the field during construction.

The above bearing capacity is provided for an estimated total maximum settlement of 25 mm. The settlement of spread footings made on the bedrock is elastic, linear and non- recoverable. The settlement occurs as load is applied. There have been a number of load tests carried out in the sound Georgian Bay Formation that have indicated that the rock formation has predictable and similar response to loading over its area of occurrence. These tests have yielded parameters to estimate the elastic compression of the rock under applied loading. This compression is a function of the pressure applied and the size of the area loaded. To estimate the settlement of foundations of different sizes and assess differential settlement between foundation units, the following relationship can be used.

$$\boldsymbol{\delta = 1000q_{SLS} \left[2 \div (1 + 0.7/B_f)\right]^2 \div k}$$

Where, δ = the estimated vertical displacement in the rock beneath the centre of the loaded footing (mm)
q_{SLS} = the applied SLS pressure on the rock at the base of the footing (kPa)
B_f = the nominal footing width (metres)
k = the modulus of displacement (kPa/m) use (400,000 kPa/m for partially weathered bedrock)

This maximum allowable net bearing capacity must be re-evaluated once the bedrock subgrade is exposed and examined by a qualified geotechnical engineer to ensure that the founding bedrock subgrade is consistent with the design bearing pressure intended by the geotechnical engineer.

5.1.1 Foundation Installation

Prior to pouring concrete for the footings, the footing subgrade must be cleaned of all deleterious materials such as softened, disturbed or caved materials, as well as any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided. As per the Ontario Building Code (2012), the foundation excavations must be inspected and approved (by Terraprobe) to ensure the bearing capacities stated below are applicable. If incompetent soils are encountered at the proposed bearing depths during foundation excavation or due to inadequate dewatering, sub-excavation to competent soil subgrade is required under the direction of the geotechnical engineer.

The underside of footing/grade beam/pile cap elevations must be designed to provide a minimum of 1.2 m of soil cover or equivalent insulation to the foundation subgrade for frost protection considerations in unheated areas. All footings must be designed to bear at least 0.3 m into the partially weathered shale.

Foundations placed directly on bedrock should be established on a relatively level rock surface, i.e. generally sloping at an angle of less than approximately 10 degrees from the horizontal. In some instances, foundation bases can be placed on bedrock sloping at angles up to 25 to 30 degrees from the horizontal, provided dowels are incorporated to resist shear. Where rock slopes are at steeper angles, the rock surface is to be levelled to provide a stepped footing base. As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide level benching over these areas by pouring lean concrete (compressive strength to be provided by the structural engineer) prior to constructing the foundations. This determination is made on site based on site specific bedrock conditions.

5.2 Earth Design Parameters

The average values for use in the design of structures subject 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 (degree)	K_a	K_o	K_p
Earth Fill	19.0	28	0.36	0.53	2.77
Clayey Silt Till	21.0	32	0.31	0.47	3.25
Georgian Bay Formation Bedrock	25.0	26	N/A	N/A	N/A

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

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

Where:
P = the horizontal pressure (kPa)
K = the earth pressure coefficient
h = the depth below the ground surface (m)
h_w = the depth below the ground water level (m)
γ = the bulk unit weight of soil (kN/m^3)
γ_w = the bulk unit weight of water (9.8 kN/m^3)
γ' = the submerged unit weight of the exterior soil, (γ_{sat} - γ_w)
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:

$$P = K[\gamma h + q]$$

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.

Resistance to sliding of the structures is developed by friction between the base of the footing and the bedrock. 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**. The coefficient of friction angle between the underside of a cast-in-place concrete footing and a partially weathered bedrock surface may be taken as **tan 26° (0.5).**

5.3 Earthquake Design Parameters

The Ontario Building Code (OBC) stipulates the methodology for earthquake design analysis. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification.

Under Ontario Regulation 88/19, the ministry amended Ontario's Building Code (O. Reg 332/12) to further harmonize Ontario's Building Code with the 2015 National Codes. These changes will help reduce red tape for businesses and remove barriers to interprovincial trade throughout the country. The amendments are based on code change proposals the ministry consulted in 2016 and 2017. The majority of the amendments came into effect on January 1, 2020, which includes structural sufficiency of buildings to withstand external forces and improve resilience.

Seismic hazard is defined in the Ontario Building Code by uniform hazard spectra (UHS) at spectral coordinates of 0.2s, 0.5s, 1.0s and 2.0s and a probability of exceedance of 2% in 50 years. The OBC method uses a site classification system defined by the average soil/bedrock properties (e.g., shear wave velocity (Vs), Standard Penetration Test (SPT) resistance, and undrained shear strength (Su) in the top 30 meters of the site stratigraphy below the foundation level, as set out in the Ontario Building Code. There are 6 site classes from A to F, decreasing in ground stiffness from A, hard rock, to E, soft soil; with site class F used to denote problematic soils (e.g., sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain peak ground acceleration (PGA), peak ground velocity (PGV) site coefficients Fa and Fv, respectively, used to modify the UHS to account for the effects of site-specific soil conditions.

Based on the above noted information, it is recommended that the site designation for seismic analysis be **Site Class B**, as per the Ontario Building Code. Consideration may be given to conducting a site-specific Multichannel Analysis of Surface Waves (MASW) at this site to confirm the average shear wave velocity in the top 30 metres of the site stratigraphy.

The values of the site coefficient for design spectral acceleration at period T, F(T), and of similar coefficients F(PGA) and F(PGV) shall conform to Tables 4.1.8.4.B. to 4.1.8.4.I. using linear interpolation for intermediate values of PGA.

5.4 Basement Floor Slab

The following subgrade parameters are recommended for the design of floor slab resting on an aggregate drainage layer overlying the cohesionless soil deposit or weathered shale:

K_s = 40,000 kPa/m (clayey silt till)
K_s = 60,000kPa/m (partially weathered bedrock)

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 sump which ensures the performance of this system must have a duplexed pump arrangement for 100% pumping redundancy and this pump must be provided with emergency power as needed. Basement and subfloor drainage provisions are further discussed in Section 5.5 of this report.

5.5 Basement Drainage

The ground water levels measured on August 28, 2020, August 3, 2022, September 6, 2022, and October 13, 2022 in the monitoring wells varied from about 1.1 m (Borehole 4) to 3.5 m (Borehole 5) depth below grade (Elev. 136.1 to 138.7 m).

To assist in maintaining basement dry from seepage, it is recommended that exterior grades around the building be sloped away at a 2 percent gradient or more, for a distance of at least 1.2 m.

The basement wall (for basements) in case of open excavation must be provided with damp-proofing provisions in conformance to the Section 9.13.2 of the Ontario Building Code (2012). The basement wall backfill for a minimum lateral distance of 0.6 m out from the wall should consist of free-draining granular material (OPSS.MUNI 1010 Granular B). Perimeter foundation drains should be provided, consisting of perforated pipe with filter fabric (minimum 100 mm diameter) surrounded by a granular filter (minimum 150 mm thick), and freely outletting. The granular filter should consist of 19 mm Clear Stone (OPSS.MUNI 1004) surrounded by a filter fabric (Terrafix 270R or equivalent). Alternatively, perimeter foundation drains are provided with a prefabricated drain material (for instance, CCW MiraDRAIN 6000 series or Terrafix Terradrain 600). The perimeter drain installation and outlet provisions must conform to the plumbing code requirements.

In case of a shored excavation along the southern limit of the existing building, prefabricated drainage composites, such as CCW Miradrain 6000 series, Terrafix Terradrain 200 or Delta-Drain 6000 HI-X, should be incorporated between the shoring wall and the cast-in-place concrete foundation wall to make a drained cavity. Drainage from the cavity must be collected at the base of the wall in non-perforated pipes and conveyed directly to the sumps. The flow to the building storm water sump from the subsurface drainage will be governed largely by the building perimeter drainage collection during rainfall and runoff events. Typical shored excavation drainage details are provided in Figure 5.

It is recommended that the subfloor drainage system consists of minimum 100 mm diameter perforated pipes. The pipes must be surrounded on all sides by a minimum of 100 mm of 19 mm clear stone, and the pipe inverts (placed in trenches) should be a minimum 300 mm below the base course of the slab. The elevator pits can be drained separately with an independent lower pumping sump or can be designed as water proof structures which are below the drainage level. The alternative to a 200 mm drainage layer and trenched subfloor drainage is to cut the rock subgrade neat to 300 mm beneath the floor slab and place subdrains directly on the subgrade. The subfloor drainage layer then comprises 300 mm thick layer of 19 mm clear stone (OPSS.MUNI 1004).

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). The sub-drain installation and outlet must conform to the plumbing code requirements.

5.6 Excavation and Ground Water Control

The boreholes data indicate that the earth fill/granular fill 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 fill materials encountered in the boreholes are classified as Type 3 Soil and the glacial till deposit encountered in the borehole is classified as Type 2 Soil above and Type 3 below the prevailing ground water level under these regulations 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 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.

The bedrock below the site, while predominantly shale, contains beds of harder limestone. It is possible that some thick layers of hard limestone may be encountered when excavations extend into the bedrock. The Georgian 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. Excavating detailed shapes for foundations and the edges of the excavation are normally accomplished with hoe mounted hydraulic rams. The ability to remove the rock in a vertical face without over excavation and dislodging of additional rock is largely dependent on the skill of the machine operator. Where an excavation must be made neat such as beside an existing footing, then greater certainty for the line of excavation can be provided by line drilling the rock with series of close spaced vertical holes (100 mm diameter 300 mm on centre) this makes a break line for the excavation in a vertical plane.

Hard layers of limestone within the shale formation are normally broken with hoe mounted hydraulic rams before excavation. Where a harder layer coincides with the foundation level, it may be necessary to remove the entire thickness of the hard layer to expose the founding level. It is virtually impossible to remove a portion of one of these layers. This can result in excess rock removal not intrinsic to the project requirements. The risk and responsibility for the excess rock removal under these circumstances and the supply and placement of the extra concrete to restore the foundation grade must be addressed in the contract documents for foundations, excavation, and shoring contractors.

The ground water levels measured on August 28, 2020, August 3, 2022, September 6,2022, and October 13,2022 in the monitoring wells were varied from about 1.1 m (Borehole 4) to 3.5 m (Borehole 5) depth below grade (Elev. 136.1 to 138.7 m).

The earth fill material at the site may contain perched water that will seep into excavations in the short term. It is expected that trapped ground water zones, are of limited extent and can be allowed to drain into the excavation, to be pumped out. This may take time, and therefore, the issues of delay in excavation must be addressed in the excavation contract. In general, the volume of ground water to be anticipated to flow into open excavations is such that temporary pumping from the excavations is expected to suffice for the control of the ground water.

As noted above, there should be limited seepage from the bedrock and overburden, as the overburden comprises a relatively low permeability till and the fracture permeability of the rock is low and diminishes with increasing depth. However, there may be water taking required during the construction phase from the water seepage emanating from the perched ground water present in the earth fill and/or permeable lenses and layers typically found within the glacial till deposit as well as water accumulated due to precipitation.

Although it is not anticipated, 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.

The Ministry of the Environment, Conservation and Parks (MECP) has recently made changes to the requirement for Permit to Take Water approvals for construction related activities. Under the revised requirements, specific construction-related water-taking activities are eligible for Environmental Activity and Sector Registry (EASR). The trigger volume for EASR registration is water taking of more than 50,000 litres/day. This includes the ground water that is collected in the open excavation as well as any precipitation and/or surface runoff that enters the excavation. Further guidance regarding dewatering will be required based on the design excavation depths.

5.7 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 will likely be wet zones within the subsurface soils (particularly soils excavated from below the prevailing water level) 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 SPMDD at a water content close to optimum (within 2 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.

5.8 Shoring Design

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 existing building has no basement while the proposed building would include the one-level underground parking structure, extending deeper than the existing building foundations. In addition, the proposed and existing buildings are connected along the southern limit of the existing building. Therefore, a special attention should be made along the proposed excavation shoring sections adjacent to the southern limit of the existing building. No excavation shall extend below a line cast as one vertical to one horizontal from foundations of the existing structure without adequate alternate support being provided. The underpin details are provided in Figure 6. A foundation which lies above a line drawn upward at 10 horizontal to 7 vertical from the excavation is within the zone of potential influence of excavation, and support for the existing foundations must be carefully assessed and augmented. The sections along the other excavation limits are about 7 to 20 m away from the property limits, therefore, open excavation may be carried out at these sections.

The shoring requirements for the site will have to be examined in detail with respect to the site boundary constraints, once the development details and the building footprint is finalized. Depending upon the boundary conditions, structures in the vicinity and the project design, the shoring system may consist of a rigid (interlocking drilled caissons) or a steel soldier piles and timber lagging shoring system.

5.8.1 Earth Pressure Distribution

Applicable soil parameters are included in the Earth Pressure Design Parameters Section (Section 5.3).

A single level of support will likely be required for shoring system. 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 = the total depth of excavation (m)
γ = the bulk unit weight of soil (kN/m^3)
q = the complete surcharge loading (kPa)

5.8.2 Caisson Toe Design

Caisson pile toes will be made in bedrock of the Georgian Bay Formation along the southern limit of the existing building where the proposed building is connected. The factored ultimate vertical bearing capacity for the design of a pile, embedded in the sound bedrock, is 10 MPa. The maximum factored ultimate lateral geotechnical resistance of the sound rock at ULS is 1 MPa.

It should be noted that possibility exists that there can be zones of material in the subsurface soils which could be sufficiently wet and permeable (specially fill materials) such that augered borings for soldier piles made through these soils could be unstable. In these cases, it will be necessary to advance temporarily cased holes to prevent caving and facilitate soldier pile installation.

The exposed Georgian Bay Formation deteriorates with time. Exposed excavation faces have been found to flake and recede as much as 300 mm with 12 months exposure. This recession generally takes the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Typically the soldier piles advanced as part of the caisson wall are advanced at least 1 meter below the base of the excavation to accommodate this weathering, to ensure that the lateral and vertical bedrock capacity provided in the design can be achieved.

5.8.3 Shoring Support

The rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented.

The shoring system should be supported by pre-stressed anchors extending beneath the adjacent lands. Pre-stressed anchors are installed and stressed in advance of excavation to limit movement of the shoring system as much as is practically possible. The use of anchors on adjacent properties requires the consent of the adjacent land owners, expressed in encroachment agreements.

Conventional earth anchors could be made with a continuous hollow stem augers or alternatively post-grouted wash bored anchors can be made. The design adhesion for earth anchors is controlled as much by the installation technique as the soil and therefore a proto-type anchor must be made in each anchor level executed to demonstrate the anchor capacity (performance tested) and validate the design assumptions. All production anchors must be proof-tested to 133% of the design load, to validate the design assumptions.

Conventional rock anchors made in bedrock of the Georgian Bay Formation may be designed using a working adhesion of 620 kPa. Earth anchors in the clayey silt till deposit are generally avoided due to potential creep. Above bond capacity is provided for preliminary design guidance and must be confirmed by onsite full-scale testing. It is imperative that all borings made for anchorages be completely cased as the holes are advanced and that there is no excess removal of soil beyond the volume of the boring made.

Where anchors cannot be used, internal bracing or rakers would be necessary. The footings for the rakers would be made on the sound bedrock where they could be designed using a maximum geotechnical resistance at ULS of 2,000 kPa when inclined at 45 degrees.

Final shoring design recommendations will be provided based on the specific shoring design to be selected for the project.

5.9 Site Work

The bedrock at this site will become disturbed and may lose its integrity when subjected to traffic, particularly when wet. It can be expected that a bedrock subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic, especially during periods of wet weather. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed subgrade and use of granular fill material for site restoration or underfloor fill that is not intrinsic to the project requirements.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soil and rock at this site is susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.

5.10 Quality Control

The proposed structures will be founded on spread footing foundations made on unweathered (sound) bedrock of the Georgian Bay Formation. The foundation installations must be reviewed in the field by Terraprobe geotechnical engineer as they are constructed and to ensure that the subgrade remains in its original condition (undisturbed). The on-site review of the condition of the foundation bedrock 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. If Terraprobe is not retained to carry out foundation engineering field review 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 contained in this report.

The long term performance of the slab on grade is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved as much as practically possible. The design advice in this report is based on an assessment of the subgrade support capabilities as indicated by the boreholes. These conditions may vary across the site depending on the final design grades and therefore, the preparation of the subgrade and the compaction of all fill should be monitored by Terraprobe at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

The requirements for fill placement on this project have been stipulated relative to Standard Proctor Maximum Dry Density (SPMDD). In situ determinations of density during fill and asphaltic pavement placement on site are required to demonstrate that the specified placement density is achieved. Terraprobe is a CNSC certified operator of appropriate nuclear density gauges for this work and can provide sampling and testing services for the project as necessary, with our qualified technical staff.

Concrete 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.

Terraprobe staff can also provide quality control services for Building Envelope, Roofing and Structural Steel, 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 Change 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 made 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 Luke's Dixie Senior Residence Corp. 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 Luke's Dixie Senior Residence Corp. 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
2023-03-16
PROVINCE OF ONTARIO

Abdus Sobahan, M. Eng., P. Eng.
Geotechnical Engineer

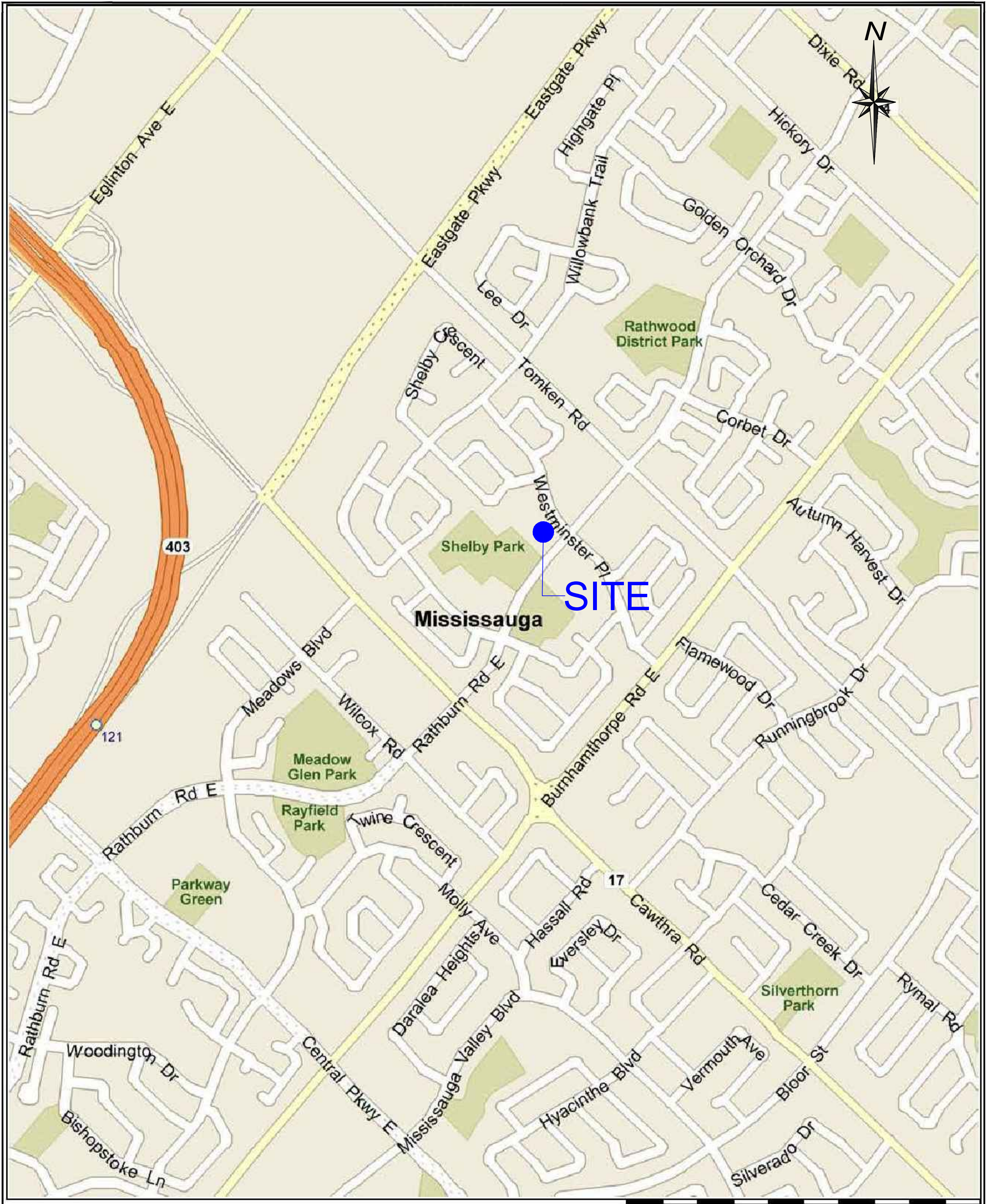
Seth Zhang, M. Eng., M. Sc., P. Eng.
Associate

ENCLOSURES

TERRAPROBE INC.

FIGURES

TERRAPROBE INC.



REFERENCE
Microsoft Streets and Trips

Terraprobe

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

Title:

SITE LOCATION PLAN

File. No.: 1-20-0258-01

FIGURE : 1

\\192.168.1.16\tss\1-Project Files\2020\1-20-0258 - 4150 Westminster Place, Mississauga\01-Geotechnical Investigation\A. Dwgs, Logs\AutoCAD\1-20-0258-01 Figure 1 & 2.dwg,
DWG To PDF.pc3, Kamal, Kamal

REFERENCE

Image ©2018 Google Earth

LEGEND

⌖ Approximate Borehole Location

SCALE 4 0 10m

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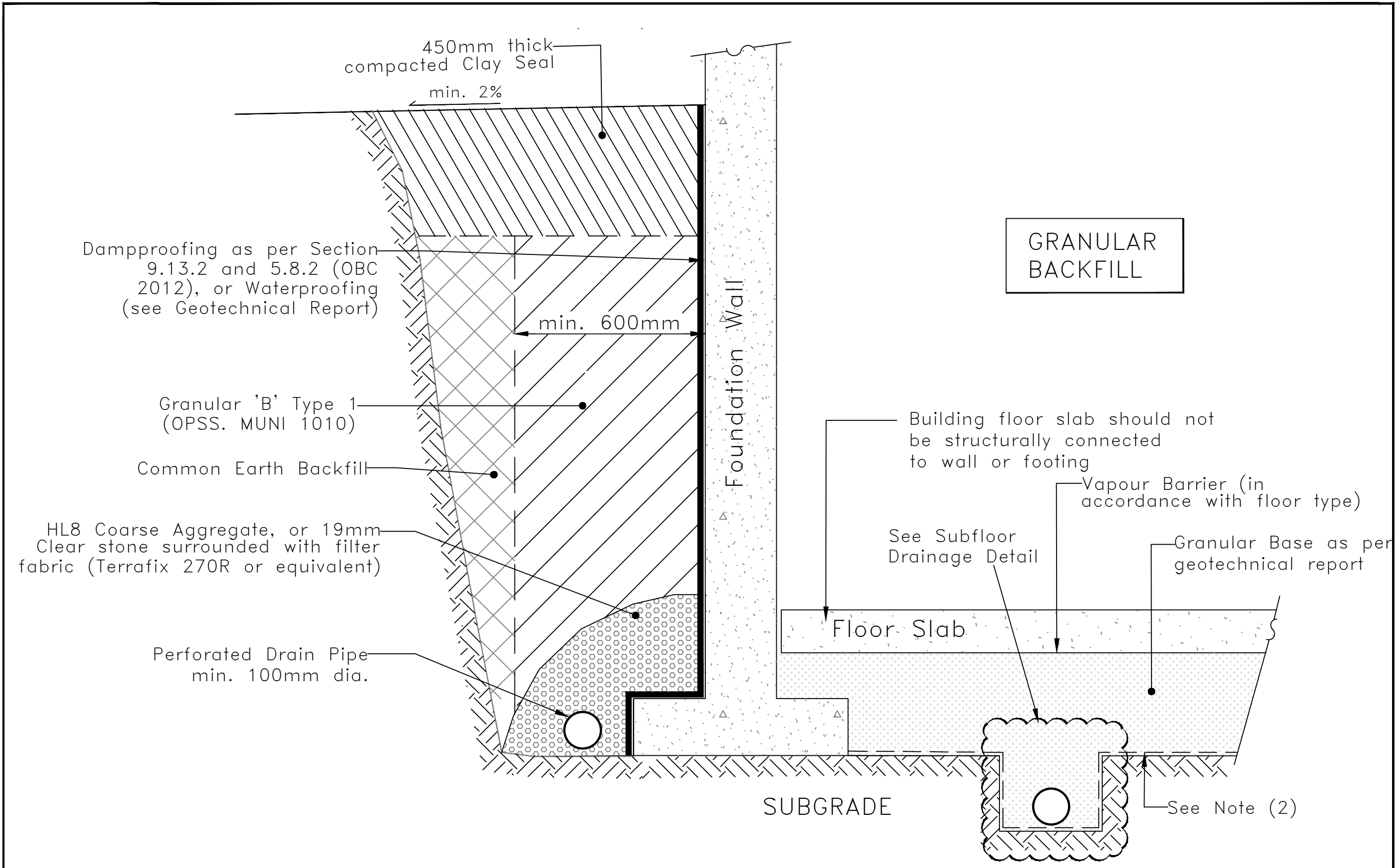
Title:

BOREHOLE LOCATION PLAN

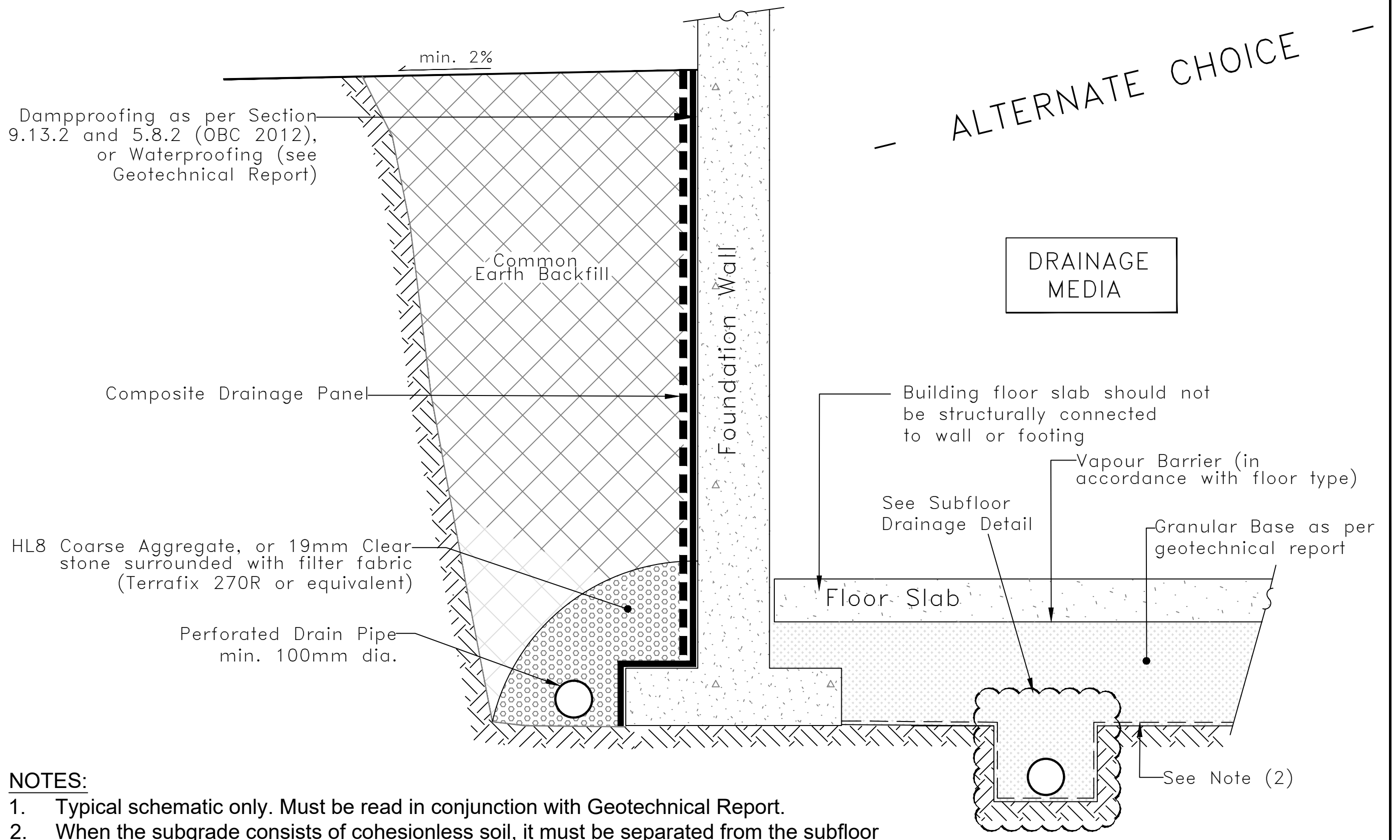
File. No.: 1-20-0258-01

FIGURE : 2

\\192.168.1.16\tss\1-Project Files\2020\1-20-0258 - 4150 Westminster Place, Mississauga\01-Geotechnical Investigation\A. Dwgs, Logs\AutoCAD\1-20-0258-01 Figure 1 & 2.dwg,
DWG To PDF.pc3, Kamal, Kamal



— ALTERNATE CHOICE —



NOTES:

1. Typical schematic only. Must be read in conjunction with Geotechnical Report.
2. When the subgrade consists of cohesionless soil, it must be separated from the subfloor drainage layer using a non-woven geotextile (Terrafix 360R or approved equivalent).
3. Not to Scale

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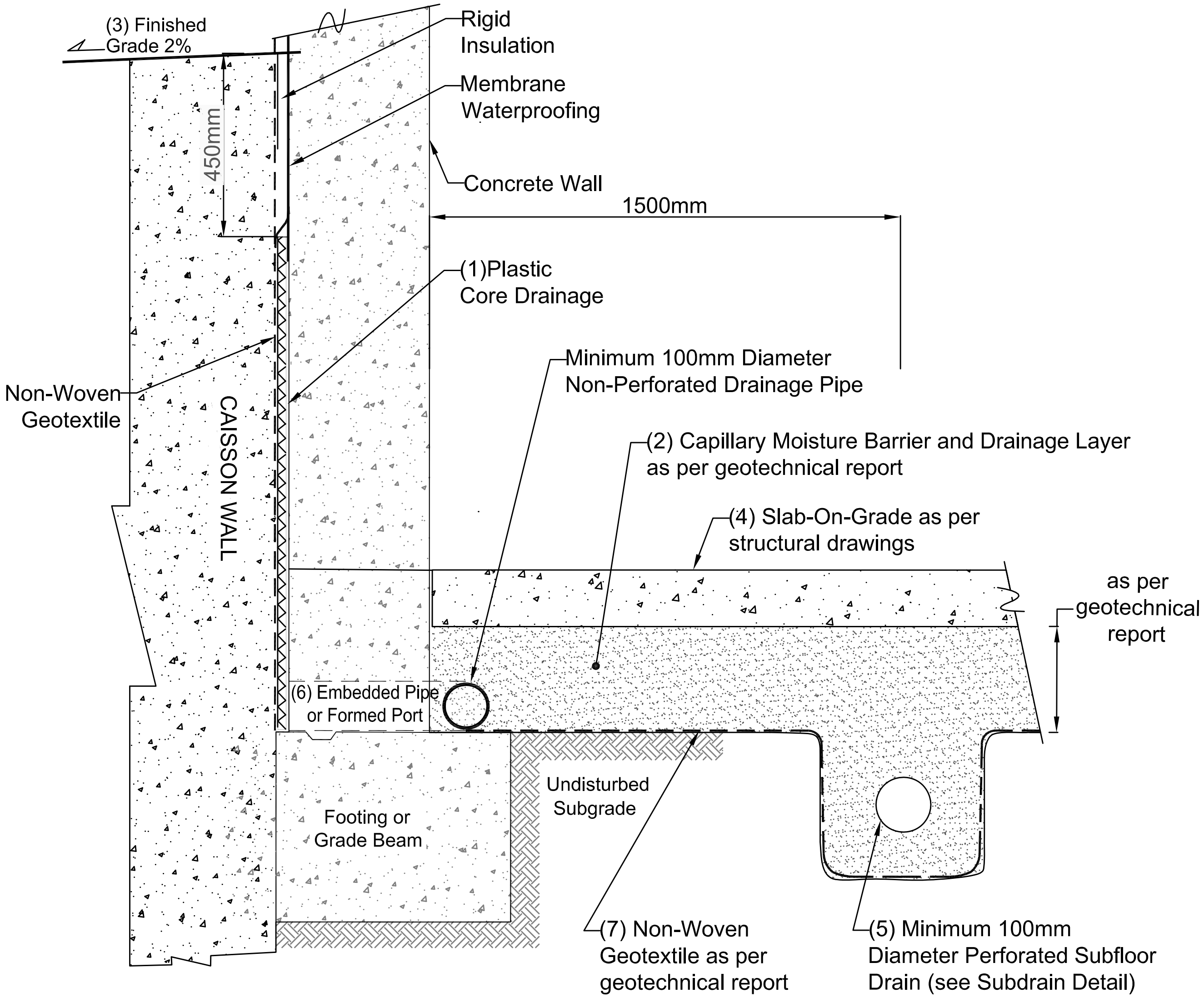
Title:

TYPICAL BASEMENT DRAINAGE SCHEMATIC
(OPEN EXCAVATION)



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.

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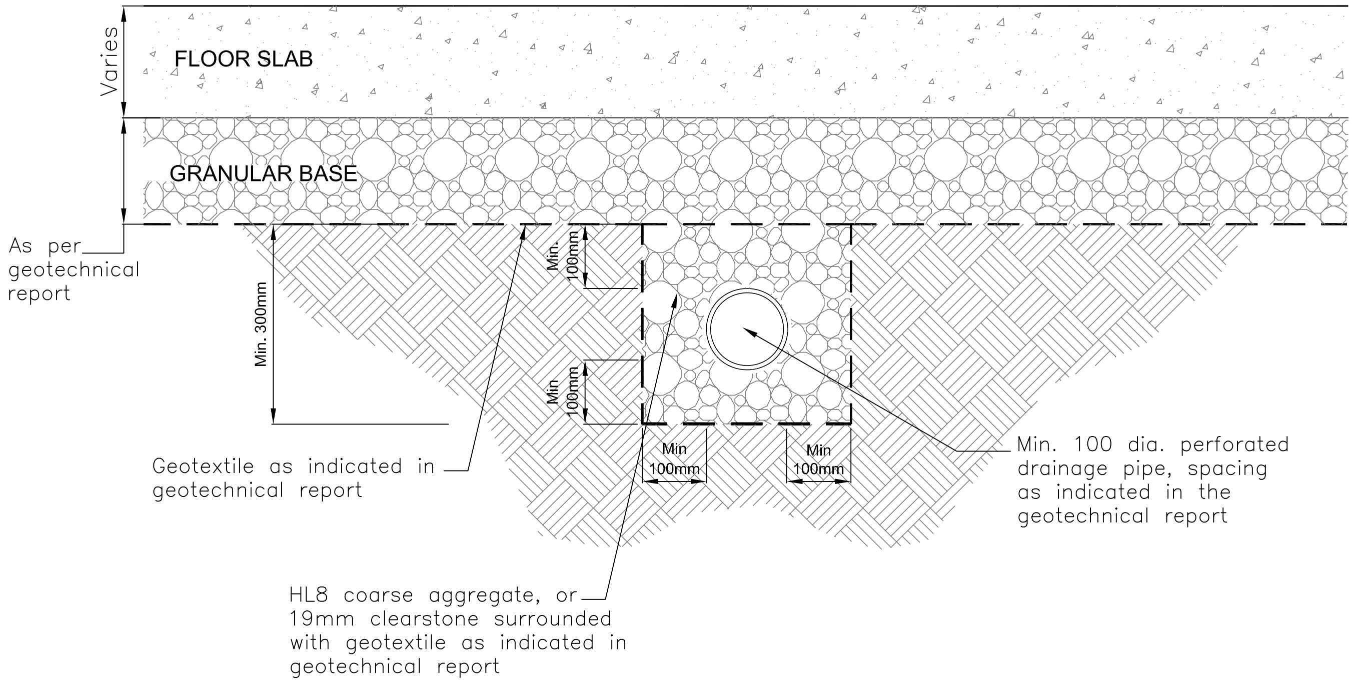
Title:

SCHEMATIC DRAINAGE DETAIL
CAISSON WALL SHORING SYSTEM



Schematic Only
Not to Scale

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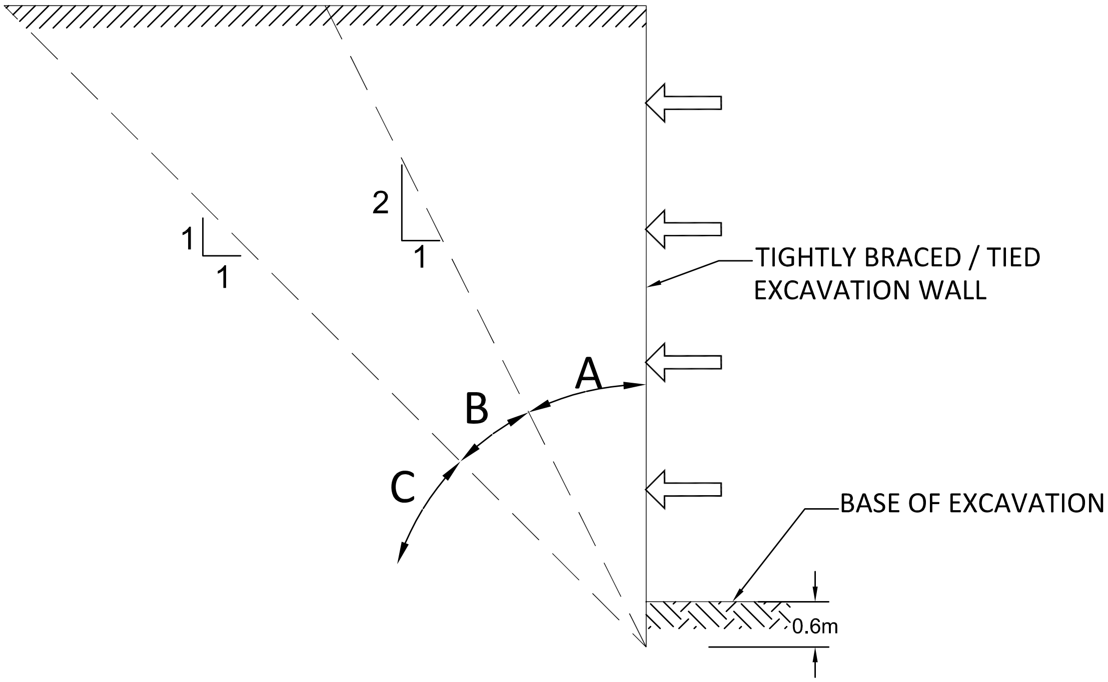
Title:

BASEMENT SUBDRAIN DETAIL



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

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Title:

GUIDELINES FOR UNDERPINNING SOILS

APPENDIX A

TERRAPROBE INC.

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ABBREVIATIONS AND TERMINOLOGY

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
silt*y*	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

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ROCK CORE TERMINOLOGY

RECOVERY

TCR **Total Core Recovery** is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

SCR **Solid Core Recovery** is the total length of sound full-diameter core pieces obtained in a core run, expressed as a percentage of the length of that core run .

RQD **Rock Quality Designation** pertains to the sum of those pieces of sound core which are 10 cm or greater in length obtained in a core run, expressed as a percentage of the length of that core run.

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
QUALITY	**very poor**	**poor**	**fair**	**good**	**excellent**

JOINT CHARACTERISTICS

Joint Spacing (adapted from *Bieniawski 1989*, *ISRM 1981*)

Classification	Spacing
very close	< 60 mm
close	60 – 200 mm
moderately close	0.2 to 0.6 m
wide	0.6 to 2 m
very wide	> 2 m

Natural Fracture Frequency (per 0.3 m)

Refers to the number of natural fractures (joints, faults, etc.) which are present per 0.3m. Ignores mechanical or drill-induced breaks, and closed discontinuities (e.g. bedding planes).

Orientation

Orientation	Angle from horiz.
horizontal/flat	0 - 20°
dipping	20 - 50°
vertical	50 - 90°

Joint Filling

Description	Approx. φ`
tight, hard, non-softening	25 - 35
oxidation, surface staining only	25 - 30
slightly altered, clay-free	25 - 30
sandy particles, clay-free	2 - 25
sandy and silty, minor clay	1 - 24
non-softening clays	6 - 12
swelling clay fillings	n/a

Joint Aperture

Classification	Aperture
closed / tight	< 0.5 mm
gapped	0.5 to 10 mm
open	> 10 mm

Planarity

- Planar
- Undulating
- Stepped
- Irregular
- Discontinuous

Roughness

- Very rough
- Rough
- Smooth
- Slickensided
- Polished

Coating	Description
clean	no filling
veneer	< 1 mm filling
coating / infill	> 1 mm filling

GENERAL

Degree of Weathering (*after MTO, RR229 Evaluation of Shales for Construction Projects*)

Zone	Degree	Description
Z1	unweathered	shale, regular jointing
Z2	partially weathered	angular blocks of unweathered shale, no matrix, with chemically weathered but intact shale
Z3		soil-like matrix with frequent angular shale fragments < 25mm diameter
Z4a		soil-like matrix with occasional shale fragments < 3mm diameter
Z4b	fully weathered	soil-like matrix only

Strength classification (*after Marinos and Hoek, 2001*)

Grade	Term	UCS (MPa)	Field Estimate (Description)
R6	extremely strong	> 250	can only be chipped by geological hammer
R5	very strong	100 - 250	requires many blows from geological hammer
R4	strong	50 - 100	requires more than one blow from geological hammer
R3	medium strong	25 - 50	can't be scraped, breaks under one blow from geological hammer
R2	weak	5 - 25	can be peeled / scraped with knife with difficulty
R1	very weak	1 - 5	easily scraped / peeled, crumbles under firm blow of geo. hammer
R0	extremely weak	< 1	indented by thumbnail

Bedding Thickness (*Quarterly Journal of Engineering Geology, Vol 3, 1970*)

Very thickly bedded	> 2 m	Medium bedded	200 – 600mm	Very thinly bedded	20 – 60mm	Thinly Laminated < 6mm
Thickly bedded	0.6 – 2m	Thinly bedded	60 – 200mm	Laminated	6 – 20mm	

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LOG OF BOREHOLE 1

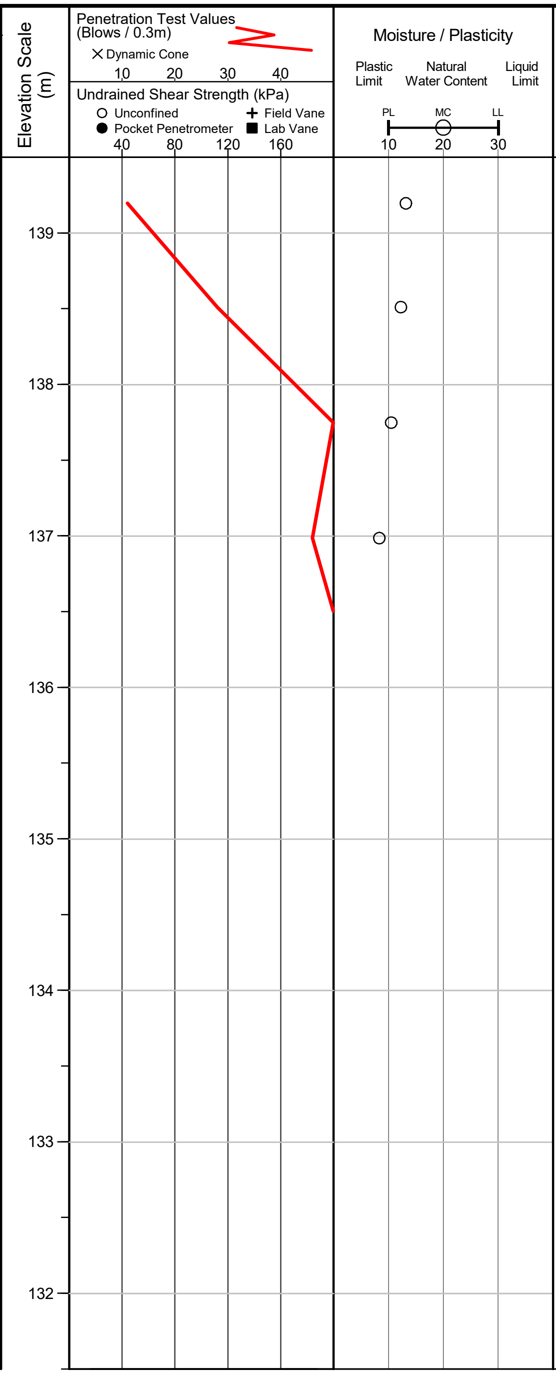
Project No. : 1-20-0258-01
Date started : July 31, 2020
Sheet No. : 1 of 1

Client : St. Luke's Dixie Senior Residence Corp
Project : 4150 Westminster Place
Location : Mississauga, Ontario

Originated by : DH
Compiled by : AS
Checked by : SZ

Position : E: 611471, N: 4829979 (UTM 17T)
Rig type : Truck-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Depth Scale (m)	SOIL PROFILE Elev Depth (m)	Description	Graphic Log	SAMPLES Number	Type	SPT 'N' Value	Lab Data and Comments GRAIN SIZE DISTRIBUTION (%) (MIT) GR SA SI CL
0	**139.5**	**GROUND SURFACE**					
	139.3 0.2	80mm **ASPHALTIC CONCRETE**					
		80mm **AGGREGATE**		1	SS	11	
	138.9 0.6	**FILL**, clayey silt, some sand, trace gravel, trace organics, stiff, dark brown, moist					
1		**CLAYEY SILT**, some sand to sandy, trace to some gravel, very stiff to hard, brown, moist (GLACIAL TILL)		2	SS	28	4 27 52 17
				3	SS	51	
2				4	SS	46	
3	136.5 3.0			5	SS	50 / 100mm	
		GEORGIAN BAY FORMATION (See rock core log for details)		1	RUN		...at 3.0m, rock cored November 4, 2020.
4				2	RUN		
5				3	RUN		
6				4	RUN		
7				5	RUN		
8	131.5 8.0						



Headspace Vapour (ppm)

Instrument Details

Unstabilized Water Level

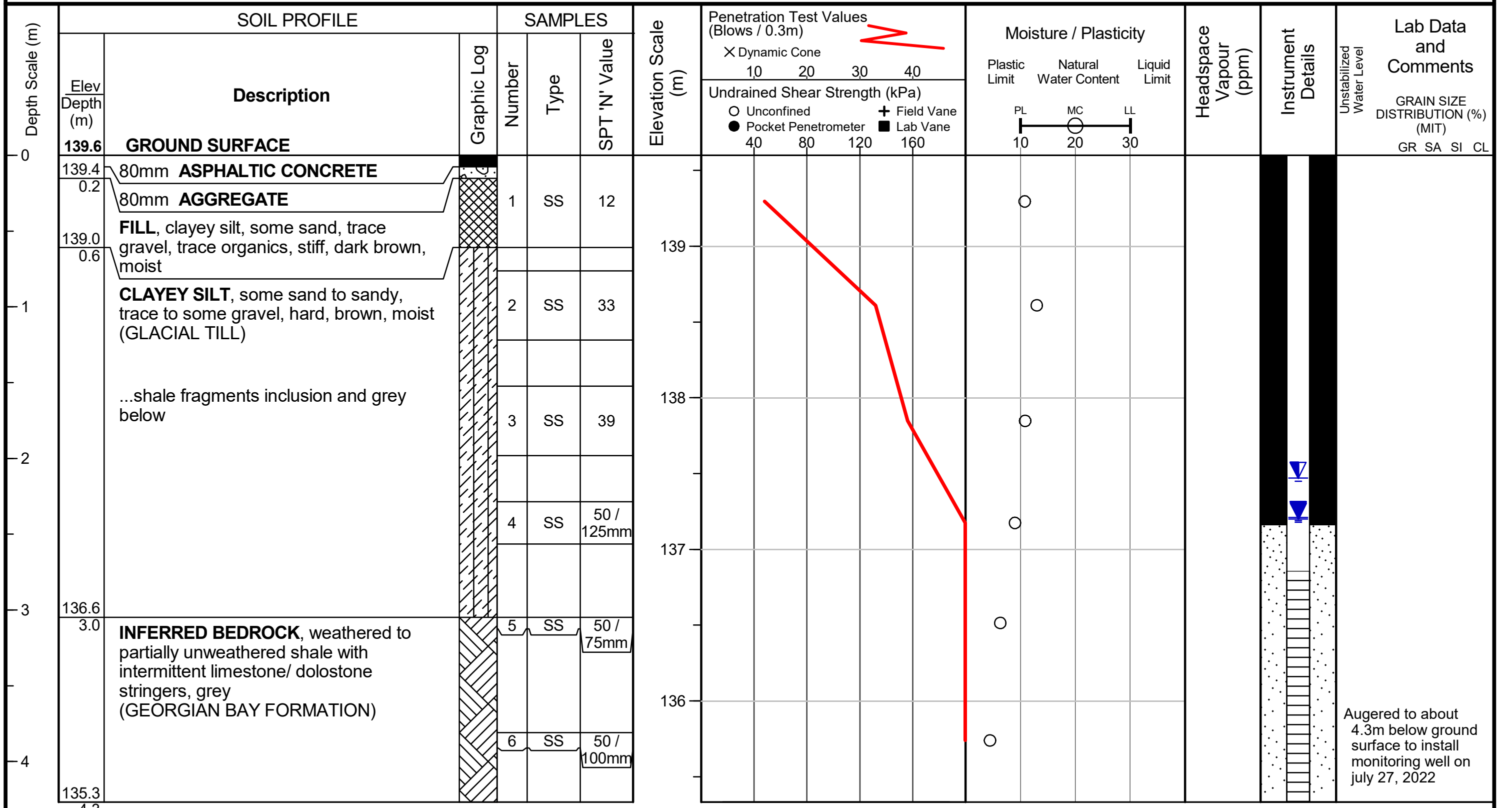
END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

LOG OF BOREHOLE 2

Project No.	: 1-20-0258-01	Client	: St. Luke's Dixie Senior Residence Corp	Originated by	: DH
Date started	: July 31, 2020	Project	: 4150 Westminster Place	Compiled by	: AS
Sheet No.	: 1 of 1	Location	: Mississauga, Ontario	Checked by	: SZ

Position : E: 611478, N: 4829990 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Truck-mounted
Drilling Method : Solid stem augers



Elev Depth (m)	Description	Number	Type	SPT 'N' Value
139.6	**GROUND SURFACE**			
139.4 0.2	80mm **ASPHALTIC CONCRETE**			
	80mm **AGGREGATE**	1	SS	12
139.0 0.6	**FILL**, clayey silt, some sand, trace gravel, trace organics, stiff, dark brown, moist			
	CLAYEY SILT, some sand to sandy, trace to some gravel, hard, brown, moist (GLACIAL TILL)	2	SS	33
	...shale fragments inclusion and grey below	3	SS	39
		4	SS	50 / 125mm
136.6 3.0	**INFERRED BEDROCK**, weathered to partially unweathered shale with intermittent limestone/ dolostone stringers, grey (GEORGIAN BAY FORMATION)	5	SS	50 / 75mm
		6	SS	50 / 100mm
135.3 4.3				

Augered to about 4.3m below ground surface to install monitoring well on july 27, 2022

END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Aug 3, 2022	2.1	137.5
Sep 6, 2022	2.4	137.2
Oct 13, 2022	2.4	137.2

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LOG OF BOREHOLE 3

Project No. : 1-20-0258-01
Date started : July 31, 2020
Sheet No. : 1 of 1

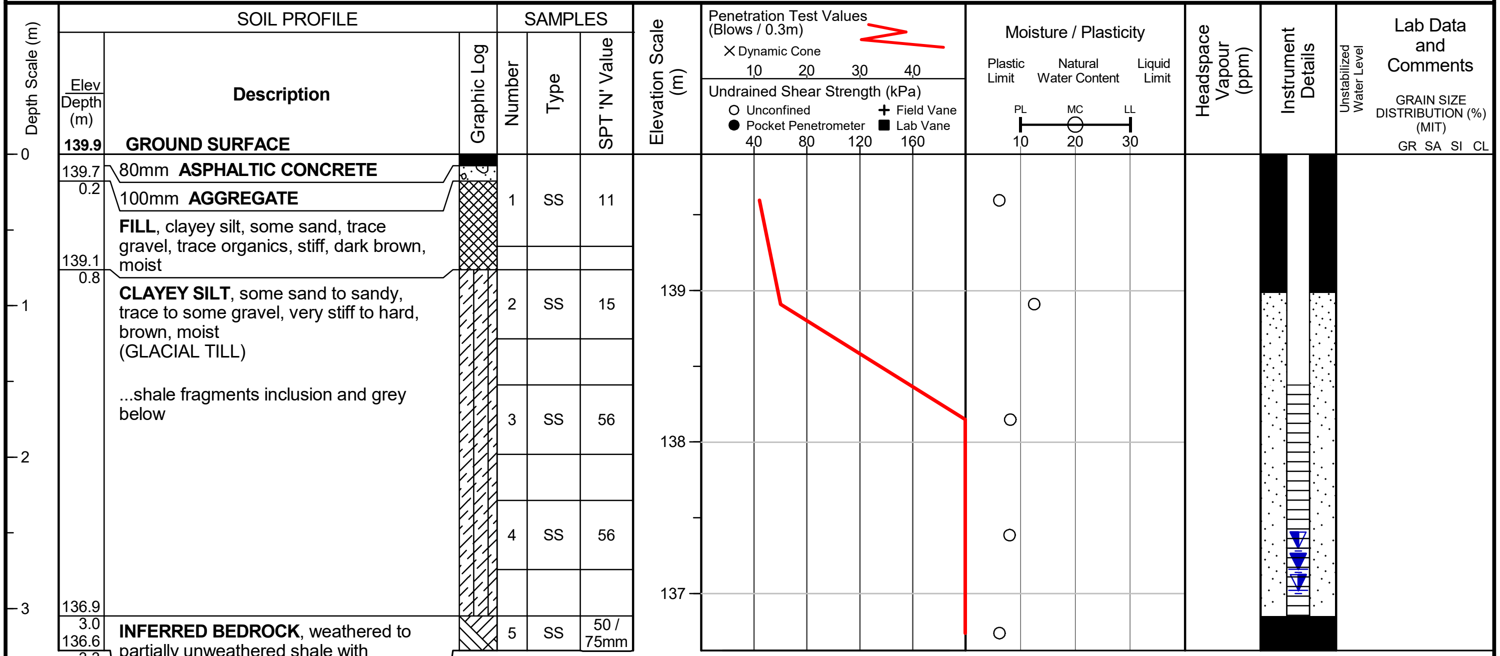
Client : St. Luke's Dixie Senior Residence Corp
Project : 4150 Westminster Place
Location : Mississauga, Ontario

Originated by : DH
Compiled by : AS
Checked by : SZ

Position : E: 611459, N: 4830007 (UTM 17T)
Rig type : Truck-mounted
Elevation Datum : Geodetic
Drilling Method : Solid stem augers

Elev / Depth (m)	Description (SOIL PROFILE)
139.9	**GROUND SURFACE**
139.7 0.2	80mm **ASPHALTIC CONCRETE**
	100mm **AGGREGATE**
139.1 0.8	**FILL**, clayey silt, some sand, trace gravel, trace organics, stiff, dark brown, moist
136.9 3.0	**CLAYEY SILT**, some sand to sandy, trace to some gravel, very stiff to hard, brown, moist (GLACIAL TILL) ...shale fragments inclusion and grey below
136.6 3.3	**INFERRED BEDROCK**, weathered to partially unweathered shale with intermittent limestone/ dolostone stringers, grey (GEORGIAN BAY FORMATION)

Number	Type	SPT 'N' Value
1	SS	11
2	SS	15
3	SS	56
4	SS	56
5	SS	50 / 75mm



WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Aug 28, 2020	2.6	137.3
Aug 3, 2022	2.9	137.0
Sep 6, 2022	2.6	137.3
Oct 13, 2022	2.7	137.2

END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

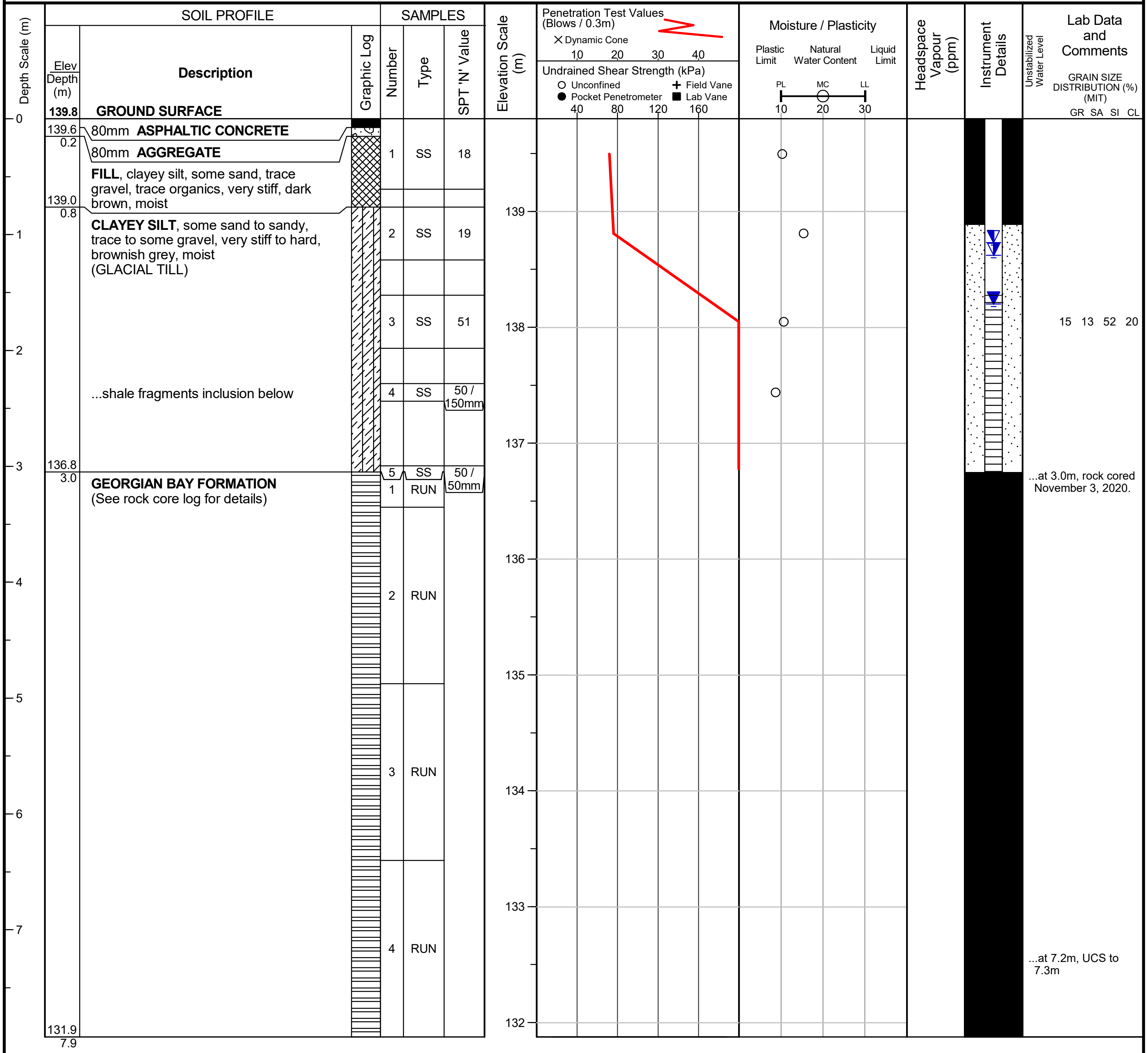
50 mm dia. monitoring well installed.

Terraprobe

LOG OF BOREHOLE 4

Project No. : 1-20-0258-01
Client : St. Luke's Dixie Senior Residence Corp
Originated by : DH
Date started : July 31, 2020
Project : 4150 Westminster Place
Compiled by : AS
Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : SZ

Position : E: 611474, N: 4830021 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Truck-mounted
Drilling Method : Solid stem augers



Elev Depth (m)	SOIL PROFILE Description	Number	Type	SPT 'N' Value
139.8	**GROUND SURFACE**			
139.6 0.2	80mm **ASPHALTIC CONCRETE**			
	80mm **AGGREGATE**	1	SS	18
139.0 0.8	**FILL**, clayey silt, some sand, trace gravel, trace organics, very stiff, dark brown, moist			
	CLAYEY SILT, some sand to sandy, trace to some gravel, very stiff to hard, brownish grey, moist (GLACIAL TILL)	2	SS	19
		3	SS	51
	...shale fragments inclusion below	4	SS	50 / 150mm
136.8 3.0		5	SS	50 / 50mm
	GEORGIAN BAY FORMATION (See rock core log for details)	1	RUN	
		2	RUN	
		3	RUN	
		4	RUN	
131.9 7.9				

Lab Data and Comments

GRAIN SIZE DISTRIBUTION (%) (MIT)

GR	SA	SI	CL
15	13	52	20

...at 3.0m, rock cored November 3, 2020.

...at 7.2m, UCS to 7.3m

END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Aug 28, 2020	1.1	138.7
Aug 3, 2022	1.2	138.6
Sep 6, 2022	1.5	138.3
Oct 13, 2022	1.6	138.2

file: 1-20-0258-01 bh logs.gpj

LOG OF BOREHOLE 5

Project No.	: 1-20-0258-01	Client	: St. Luke's Dixie Senior Residence Corp	Originated by	: DH
Date started	: July 31, 2020	Project	: 4150 Westminster Place	Compiled by	: AS
Sheet No.	: 1 of 1	Location	: Mississauga, Ontario	Checked by	: SZ

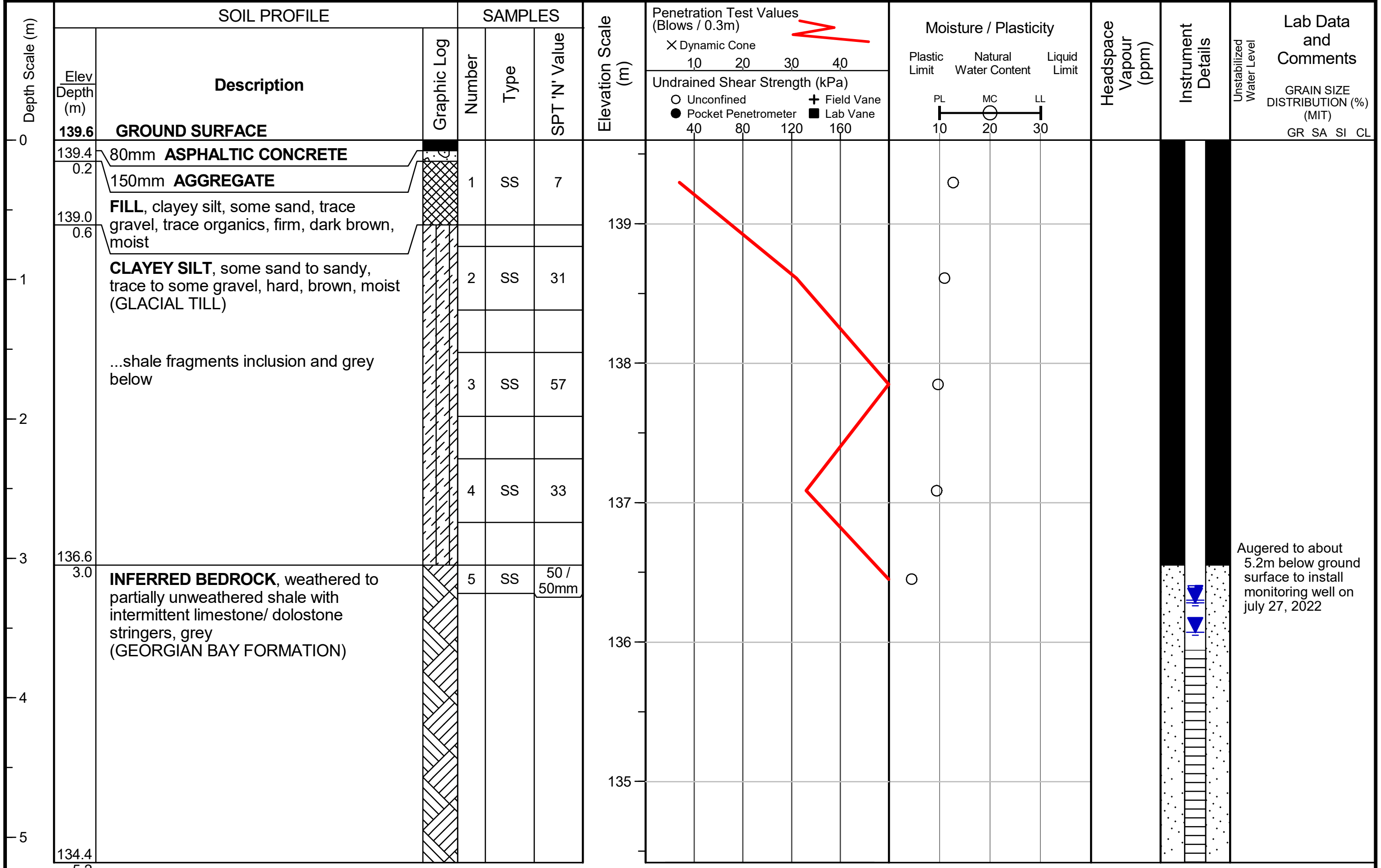
Position : E: 611488, N: 4830007 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Truck-mounted
Drilling Method : Solid stem augers

SOIL PROFILE

Elev Depth (m)	Description
139.6	**GROUND SURFACE**
139.4 0.2	80mm **ASPHALTIC CONCRETE**
	150mm **AGGREGATE**
	FILL, clayey silt, some sand, trace gravel, trace organics, firm, dark brown, moist
139.0 0.6	**CLAYEY SILT**, some sand to sandy, trace to some gravel, hard, brown, moist (GLACIAL TILL) ...shale fragments inclusion and grey below
136.6 3.0	**INFERRED BEDROCK**, weathered to partially unweathered shale with intermittent limestone/ dolostone stringers, grey (GEORGIAN BAY FORMATION)
134.4 5.2	

SAMPLES

Number	Type	SPT 'N' Value
1	SS	7
2	SS	31
3	SS	57
4	SS	33
5	SS	50 / 50mm



Augered to about 5.2m below ground surface to install monitoring well on july 27, 2022

END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Aug 3, 2022	3.3	136.3
Sep 6, 2022	3.3	136.3
Oct 13, 2022	3.5	136.1

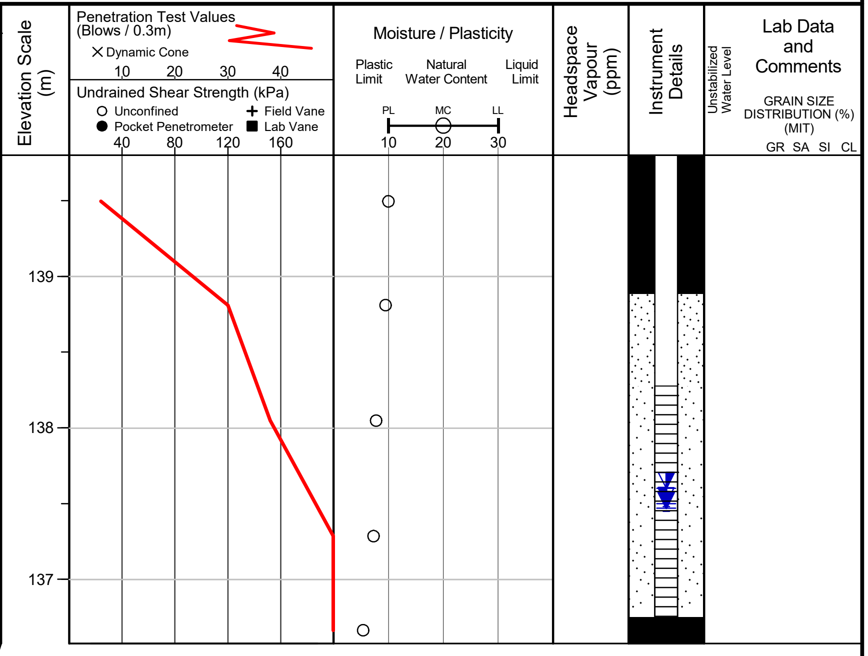
Terraprobe

LOG OF BOREHOLE 6

Project No. : 1-20-0258-01
Client : St. Luke's Dixie Senior Residence Corp
Originated by : DH
Date started : July 31, 2020
Project : 4150 Westminster Place
Compiled by : AS
Sheet No. : 1 of 1
Location : Mississauga, Ontario
Checked by : SZ

Position : E: 611439, N: 4829978 (UTM 17T)
Elevation Datum : Geodetic
Rig type : Truck-mounted
Drilling Method : Solid stem augers

Depth Scale (m)	Elev Depth (m)	SOIL PROFILE Description	Number	Type	SPT 'N' Value
0	139.8	**GROUND SURFACE**			
		65mm **CONCRETE PAVERS**, .			
		FILL, sand, trace silt, trace gravel, trace organics, loose, dark brown, moist	1	SS	6
	139.0 0.8	**CLAYEY SILT**, some sand to sandy, trace to some gravel, hard, brown, moist (GLACIAL TILL)	2	SS	30
1					
		...shale fragments inclusion and grey below	3	SS	38
2			4	SS	60
3	136.8				
	136.6 3.2	**INFERRED BEDROCK**, weathered to partially unweathered shale with intermittent limestone/ dolostone stringers, grey (GEORGIAN BAY FORMATION)	5	SS	50 / 25mm



END OF BOREHOLE
Auger refusal on inferred bedrock

Borehole was dry and open upon completion of drilling.

50 mm dia. monitoring well installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Aug 28, 2020	2.3	137.5
Aug 3, 2022	2.2	137.6
Sep 6, 2022	2.2	137.6
Oct 13, 2022	2.3	137.5

ROCK CORE LOG 1

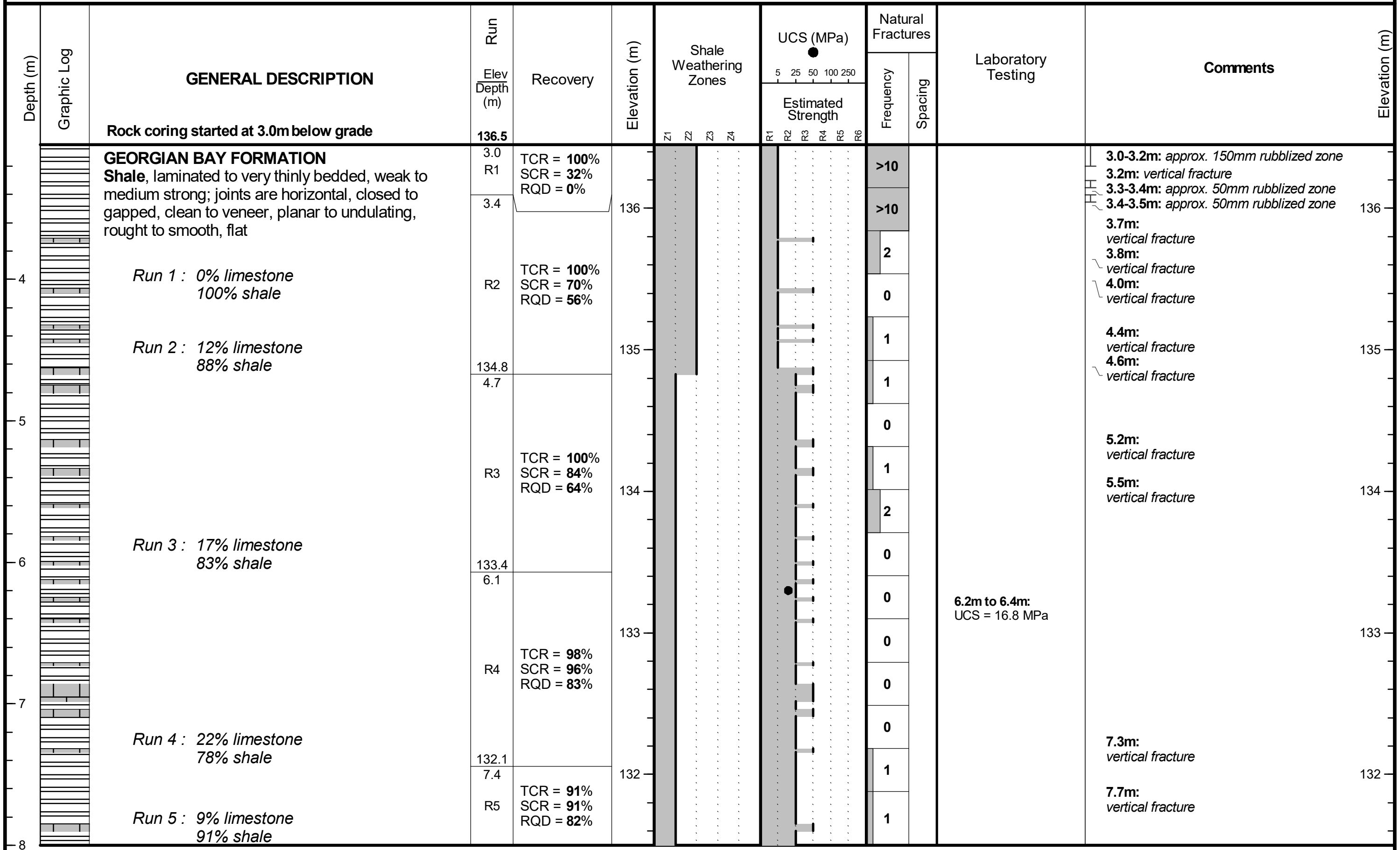
Project No. : 1-20-0258-01
Client : St. Luke's Dixie Senior Residence Corp
Originated by : DH

Date started : July 31, 2020
Project : 4150 Westminster Place
Compiled by : AS

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

Position : E: 611471, N: 4829979 (UTM 17T)
Elevation Datum : Geodetic

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



GENERAL DESCRIPTION

Rock coring started at 3.0m below grade

GEORGIAN BAY FORMATION
Shale, laminated to very thinly bedded, weak to medium strong; joints are horizontal, closed to gapped, clean to veneer, planar to undulating, rought to smooth, flat

Run 1 : 0% limestone
100% shale

Run 2 : 12% limestone
88% shale

Run 3 : 17% limestone
83% shale

Run 4 : 22% limestone
78% shale

Run 5 : 9% limestone
91% shale

END OF COREHOLE

Run	Elev / Depth (m)	Recovery
	136.5	
R1	3.0	TCR = **100%** SCR = **32%** RQD = **0%**
R2	3.4 – 134.8	TCR = **100%** SCR = **70%** RQD = **56%**
R3	4.7 – 133.4	TCR = **100%** SCR = **84%** RQD = **64%**
R4	6.1 – 132.1	TCR = **98%** SCR = **96%** RQD = **83%**
R5	7.4	TCR = **91%** SCR = **91%** RQD = **82%**
	8.0m	

Natural Fractures – Frequency: >10, >10, 2, 0, 1, 1, 0, 1, 2, 0, 0, 0, 0, 0, 1, 1

Laboratory Testing:
6.2m to 6.4m:
UCS = 16.8 MPa

Comments

3.0-3.2m: *approx. 150mm rubblized zone*
3.2m: *vertical fracture*
3.3-3.4m: *approx. 50mm rubblized zone*
3.4-3.5m: *approx. 50mm rubblized zone*
3.7m: *vertical fracture*
3.8m: *vertical fracture*
4.0m: *vertical fracture*
4.4m: *vertical fracture*
4.6m: *vertical fracture*
5.2m: *vertical fracture*
5.5m: *vertical fracture*
7.3m: *vertical fracture*
7.7m: *vertical fracture*

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ROCK CORE LOG 4

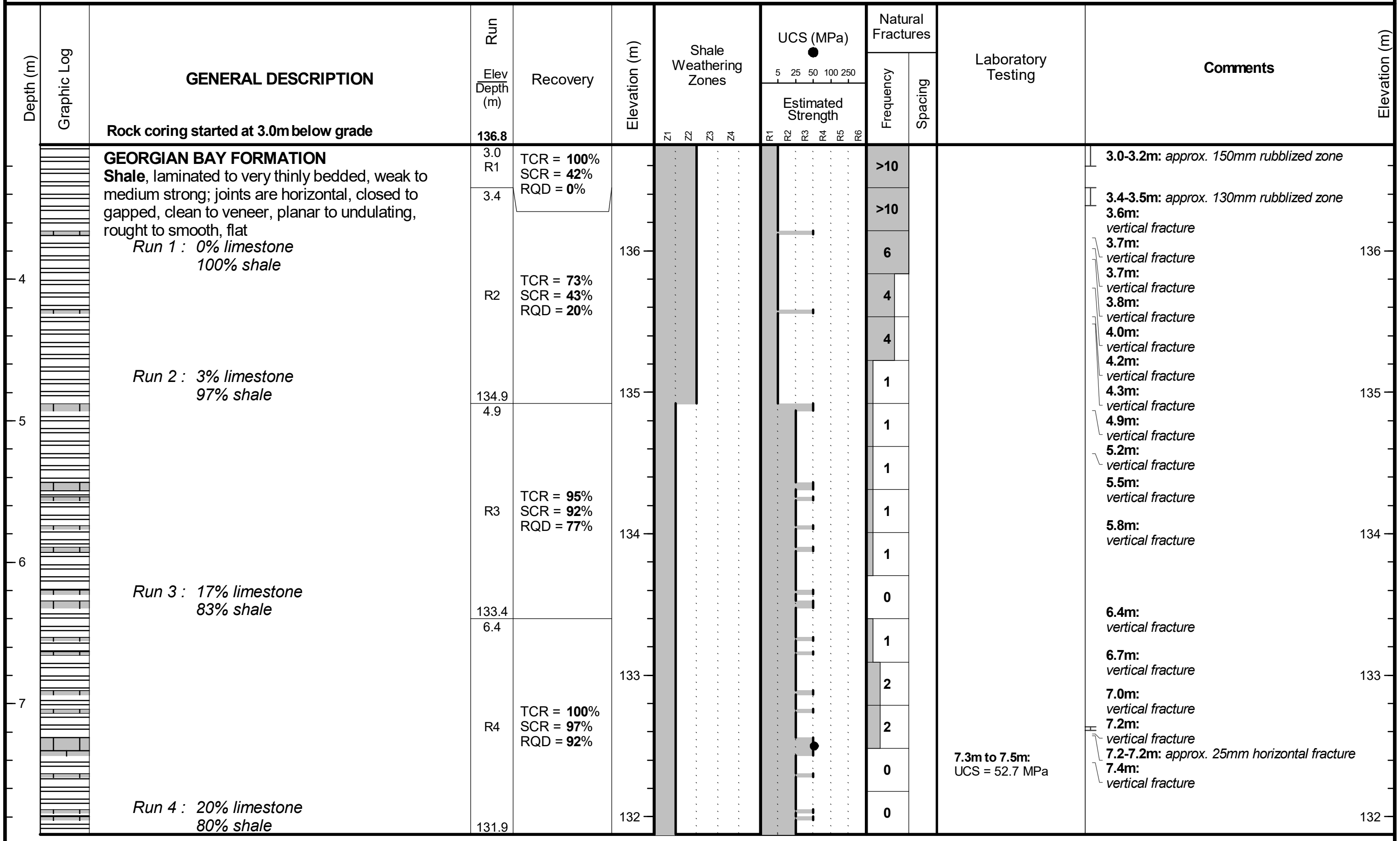
Project No. : 1-20-0258-01
Client : St. Luke's Dixie Senior Residence Corp
Originated by : DH

Date started : July 31, 2020
Project : 4150 Westminster Place
Compiled by : AS

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

Position : E: 611474, N: 4830021 (UTM 17T)
Elevation Datum : Geodetic

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



GENERAL DESCRIPTION

Rock coring started at 3.0m below grade

GEORGIAN BAY FORMATION

Shale, laminated to very thinly bedded, weak to medium strong; joints are horizontal, closed to gapped, clean to veneer, planar to undulating, rought to smooth, flat

Run 1 : 0% limestone
100% shale

Run 2 : 3% limestone
97% shale

Run 3 : 17% limestone
83% shale

Run 4 : 20% limestone
80% shale

END OF COREHOLE

Run	Elev Depth (m)	Recovery
	136.8	
R1	3.0	TCR = **100%** SCR = **42%** RQD = **0%**
R2	3.4 134.9	TCR = **73%** SCR = **43%** RQD = **20%**
R3	4.9 133.4	TCR = **95%** SCR = **92%** RQD = **77%**
R4	6.4 131.9	TCR = **100%** SCR = **97%** RQD = **92%**
	7.9m	

Laboratory Testing

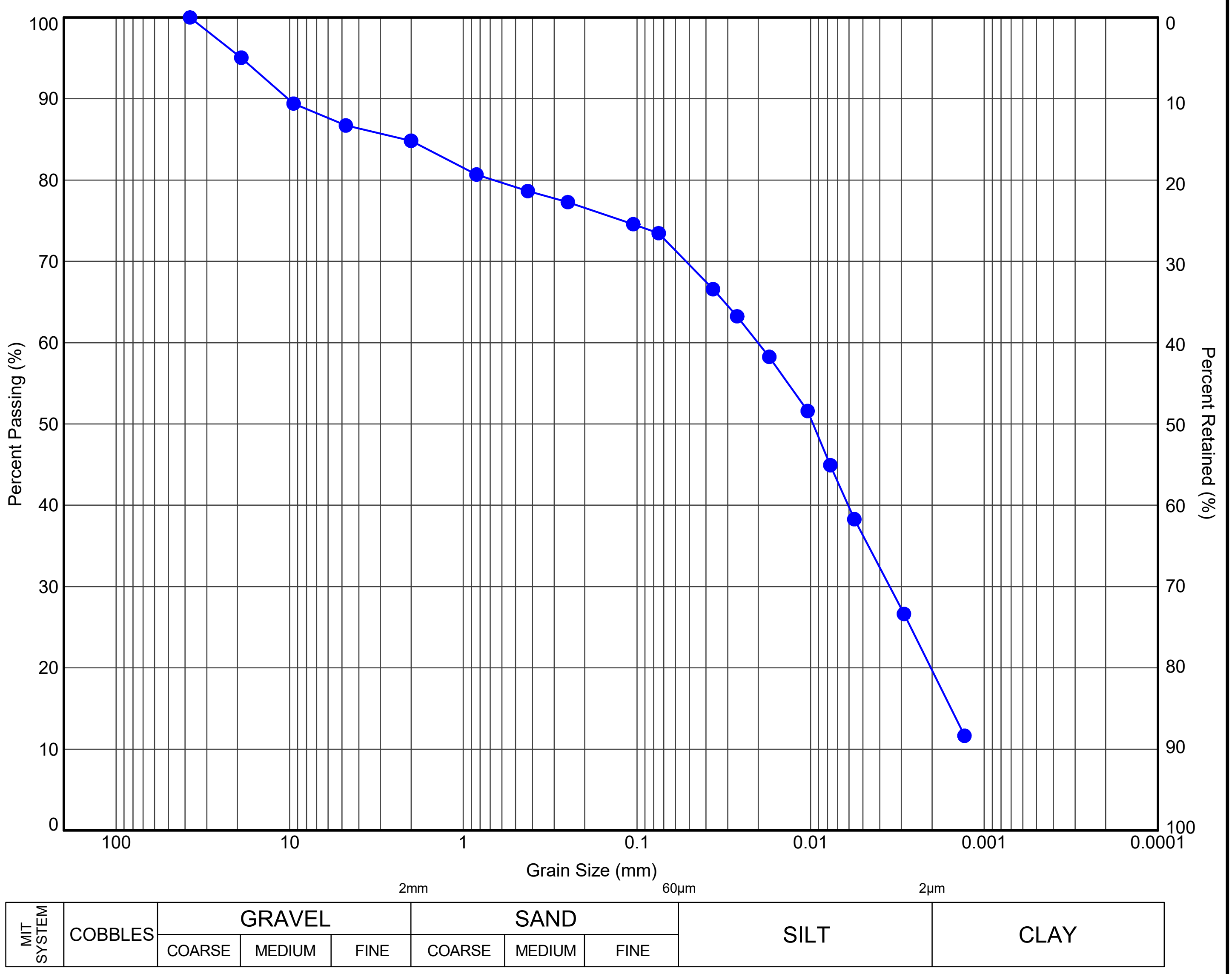
7.3m to 7.5m:
UCS = 52.7 MPa

Comments

3.0-3.2m: *approx. 150mm rubblized zone*
3.4-3.5m: *approx. 130mm rubblized zone*
3.6m: *vertical fracture*
3.7m: *vertical fracture*
3.7m: *vertical fracture*
3.8m: *vertical fracture*
4.0m: *vertical fracture*
4.2m: *vertical fracture*
4.3m: *vertical fracture*
4.9m: *vertical fracture*
5.2m: *vertical fracture*
5.5m: *vertical fracture*
5.8m: *vertical fracture*
6.4m: *vertical fracture*
6.7m: *vertical fracture*
7.0m: *vertical fracture*
7.2m: *vertical fracture*
7.2-7.2m: *approx. 25mm horizontal fracture*
7.4m: *vertical fracture*

APPENDIX B

TERRAPROBE INC.



MIT SYSTEM

	Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
●	**4**	SS3	1.8	138.0	15	13	52	20	

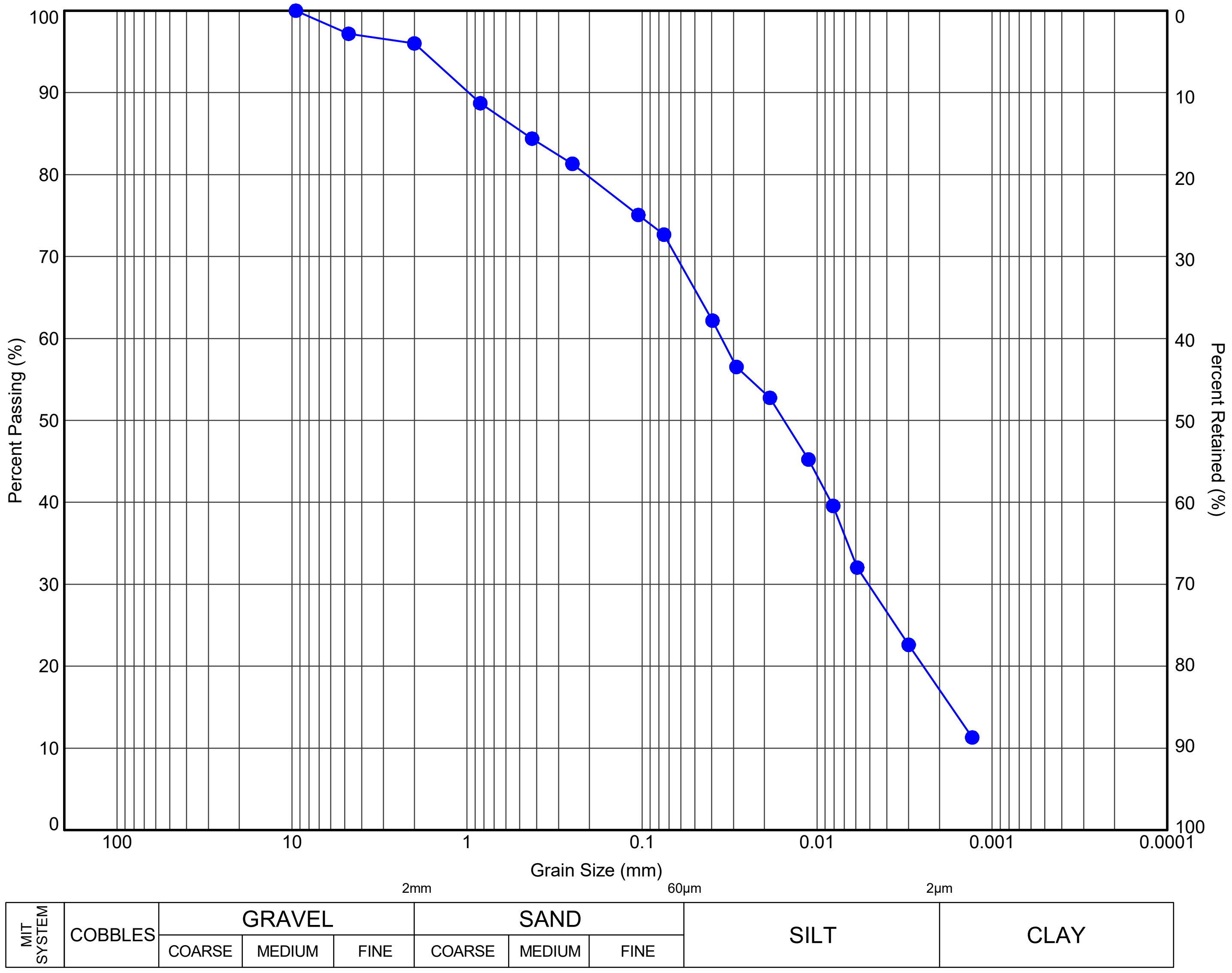
Terraprobe

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

Title:

GRAIN SIZE DISTRIBUTION
CLAYEY SILT, SOME GRAVEL, SOME SAND

File No.: **1-20-0258-01**



				MIT SYSTEM				
Hole ID	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	*(Fines, %)*
● **1**	SS2	1.0		4	27	52	17	

Terraprobe

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

Title:

GRAIN SIZE DISTRIBUTION
SANDY SILT, SOME CLAY, TRACE GRAVEL

File No.: **1-20-0258-01**

APPENDIX C

TERRAPROBE INC.

Rock Core Samples- Borehole 1
Runs: 1, 2 & 3
Depth: 3.0 to 6.1 meters

Rock Core Samples – Borehole 1
Runs: 4 &5
Depth: 6.1 to 8.0 meters

Rock Core Samples- Borehole 4
Runs: 1, 2 & 3
Depth: 3.0 to 6.4 meters

Rock Core Samples- Borehole 4
Runs: 4
Depth: 6.4 to 7.9 meters