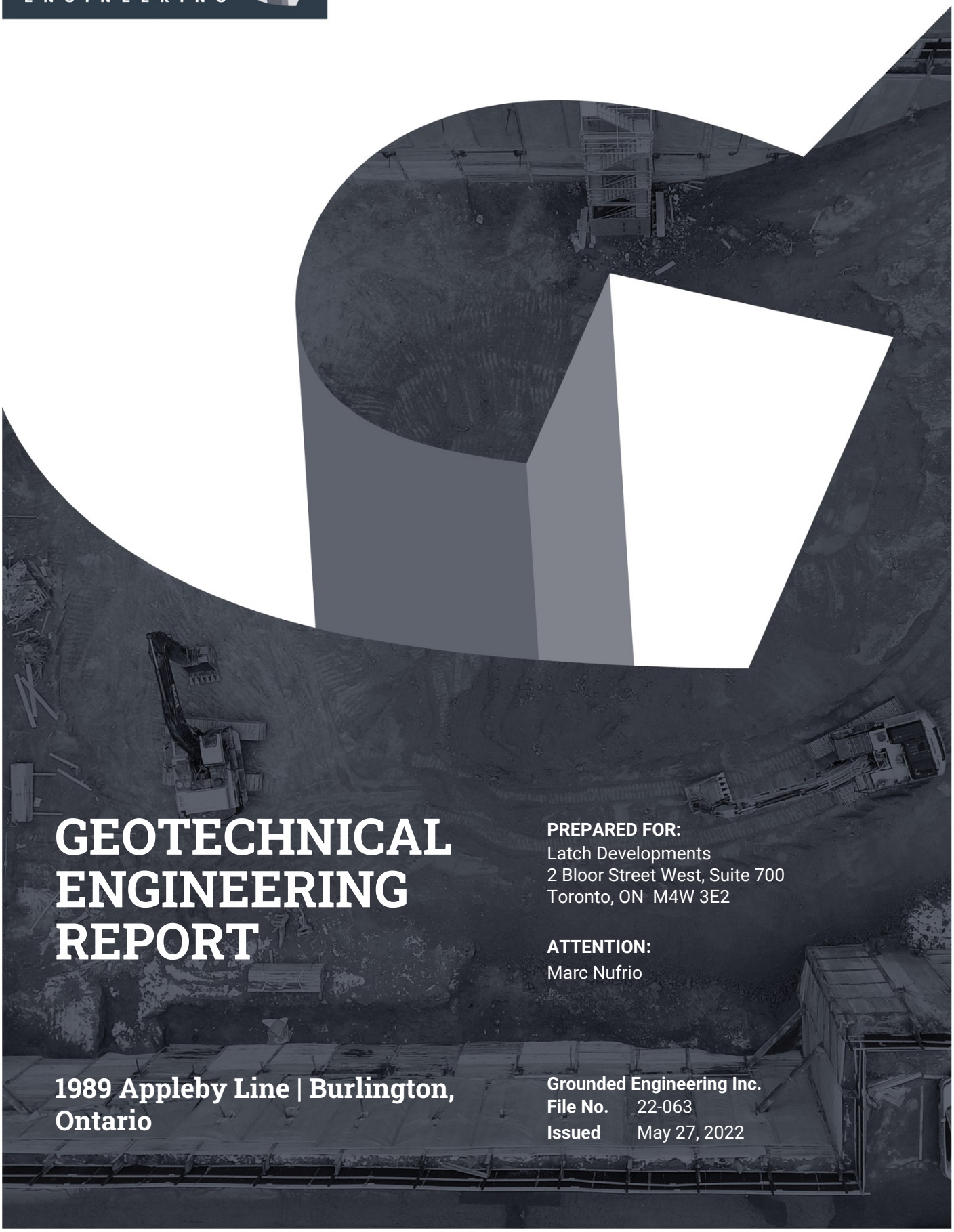


GEOTECHNICAL ENGINEERING REPORT

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1989 Appleby Line | Burlington, Ontario

Grounded Engineering Inc.
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TABLE OF CONTENTS

1 INTRODUCTION 4

2 GROUND CONDITIONS 4

2.1 SOIL STRATIGRAPHY 5

2.1.1 Surficial and Earth Fill 5

2.1.2 Native Tills 5

2.1.3 Bedrock 6

2.2 GROUNDWATER 8

2.3 CORROSIVITY AND SULPHATE ATTACK 9

3 GEOTECHNICAL ENGINEERING RECOMMENDATIONS 10

3.1 FOUNDATION DESIGN PARAMETERS 10

3.1.1 Spread Footings 11

3.1.2 Raft Foundation 11

3.2 EARTHQUAKE DESIGN PARAMETERS 13

3.3 EARTH PRESSURE DESIGN PARAMETERS 13

3.4 ROCK SWELL 15

3.5 SLAB ON GRADE DESIGN PARAMETERS 15

3.6 LONG-TERM GROUNDWATER AND SEEPAGE CONTROL 16

3.6.1 Drained Basement 16

3.6.2 Watertight Basement 17

4 CONSIDERATIONS FOR CONSTRUCTION 17

4.1 EXCAVATIONS 17

4.2 SHORT-TERM GROUNDWATER CONTROL 18

4.3 EARTH-RETENTION SHORING SYSTEMS 19

4.3.1 Lateral Earth Pressure Distribution 19

4.3.2 Soldier Pile Toe Embedment 20

4.3.3 Lateral Bracing Elements 20

4.4 SITE WORK 21

4.5 ENGINEERING REVIEW 22

5 LIMITATIONS AND RESTRICTIONS 22

5.1 INVESTIGATION PROCEDURES 22

5.2 SITE AND SCOPE CHANGES 23

5.3 REPORT USE 24

6 CLOSURE 24

FIGURES

Figure 1 – Site Location Plan
Figure 2 – Borehole Location Plan
Figure 3 – Subsurface Profile

APPENDICES

Appendix A – Grounded Borehole Logs; Abbreviations and Terminology
Appendix B – Geotechnical Laboratory Results
Appendix C – Chemical Analysis, Corrosivity Parameters
Appendix D – Rock Core Photographs
Appendix E – Rock Mechanics Laboratory Results
Appendix F – Typical Basement Drainage Details
Appendix G – Excavation Zone of Influence Guidelines

1 Introduction

Latch Developments has retained Grounded Engineering Inc. (“Grounded”) to provide geotechnical engineering design advice for their proposed development at 1989 Appleby Line, in Burlington, Ontario.

Grounded has already provided a preliminary geotechnical report for the proposed development of this site. This report replaces and supersedes that previous preliminary report.

The proposed project includes demolishing the existing gas station and constructing a 20± storey residential building, with three underground parking levels set at a lowest (P3) Finished Floor Elevation (FFE) of about 129.1± m.

Grounded has been provided with the following reports and drawings to assist in our geotechnical scope of work:

- Site survey, “Part of Lot 5, Concession 2, South of Dundas Street”; Project No. 3504-0, dated September 27, 2021, prepared by R. Avis Surveying Inc.
- Architectural Drawings – Conceptual Site Plans, “1989 Appleby Line”; Project No. 21058, received May 24, 2022, prepared by RAW Design.

Grounded’s subsurface investigation of the site to date includes five (5) boreholes (Boreholes 101 to 105) which were advanced from April 4th to 8th, 2022.

Based on the borehole findings, geotechnical engineering advice for the proposed development is provided for foundations, seismic site classification, earth pressure design, slab on grade design, basement drainage, and pavement design. Construction considerations including excavation, groundwater control, and geostructural engineering design advice are also provided.

Grounded Engineering must conduct the on-site evaluation of founding subgrade as foundation and slab construction proceeds. This is a vital and essential part of the geotechnical engineering function and must not be grouped together with other “third-party inspection services”. Grounded will not accept responsibility for foundation performance if Grounded is not retained to carry out all the foundation evaluations during construction.

2 Ground Conditions

The borehole results are detailed on the attached Grounded borehole logs. Our assessment of the relevant stratigraphic units is intended to highlight the strata as they relate to geotechnical engineering. The ground conditions reported here will vary between and beyond the borehole locations.

The stratigraphic boundary lines shown on the borehole logs are assessed from non-continuous samples supplemented by drilling observations. These stratigraphic boundary lines represent

transitions between soil types and should be regarded as approximate and gradual. They are not exact points of stratigraphic change.

Elevations are measured relative to geodetic datum based on a benchmark on the provided site survey. The horizontal coordinates are provided relative to the Universal Transverse Mercator (UTM) geographic coordinate system.

Asphalt and granular thicknesses reported here are observed in individual borehole locations through the top of the open borehole. Thicknesses may vary between and beyond the boreholes.

2.1 Soil Stratigraphy

The following soil stratigraphy summary is based on the borehole results and the geotechnical laboratory testing. A subsurface profile showing stratigraphy and engineering units is appended.

2.1.1 Surficial and Earth Fill

Borehole 101, 102 and 103 encountered a 140 mm thick asphalt pavement structure overlying an aggregate layer. Borehole 104 and 105 encountered 75 mm of topsoil.

Underlying the pavement structure, all boreholes observed a layer of earth fill that extends to depths of 1.5 meters below grade (Elev. 137.8 to 138.3 meters). The earth fill varies in composition but generally consists of silty sand to clayey silt. It contains trace asphalt, brick, rootlets, and wood fragments. The earth fill is typically brown and dark brown and moist. Due to variation and inconsistent placement of earth fill material, the relative density of the earth fill varies but is on average stiff.

2.1.2 Native Tills

Underlying the fill materials, the boreholes encountered an undisturbed native glacial till deposit that extends to depths of 3.7 to 4.3 meters below grade (Elev. 135.2 to 135.7 meters). The till has a generally clayey silt matrix that is moist, and dark brown to reddish brown. It contains some sand, and becomes sandier with depth. Immediately above the bedrock surface (i.e. within about 0.5 m above the bedrock), traces of rock (shale) fragments and cobbles were often encountered.

Standard Penetration Test (SPT) results (N-Values) carried out within the tills observed N-valued ranging from 10 to over 50 blows per 300 mm of penetration (“bpf”), indicating a relative density ranging from firm/loose to hard/very dense (on average, hard/dense). Within about 0.5 m of the top of bedrock, the higher N-Values were observed in the sandy silt till, which are likely due to the inferred cobbles, or refusal on the underlying bedrock surface.

2.1.3 Bedrock

Bedrock was confirmed by rock core recovered in all Grounded Boreholes at 3.7 to 4.3 meters below grade (Elev. 135.2 to 135.7 meters), extending beyond the depths of investigation at 14.0 to 15.4 m below grade (Elev. 123.9 to 125.8 m).

Detailed core logs are included with the corresponding borehole logs. Photographs of the recovered rock core and a guide of rock core terminology are appended. The rock core terminology sheet defines many of the descriptive terms used below.

The bedrock beneath the site is the Queenston Formation, which is a deposit predominantly comprising thin to medium bedded red to reddish brown calcareous shale and siltstone ("shale" by local convention) of Upper Ordovician age. The strength of the intact sound bedrock varies from weak to medium strong. The low-fissility shale contains grey-green mottling and banding which is occasionally harder than the red shale, possibly indicating a higher carbonate content which is called “limestone” by local convention. However, the Queenston shale within the study area is generally calcareous and is interbedded with stronger calcareous sandstone and silty bioclastic carbonate. Minor amounts of gypsum, in nodules and laminae, are found throughout. These, along with occasional weathered clay seams and partings, indicate the presence of groundwater within the bedrock.

Joints occurring within the shale are closely to very closely spaced, and typically weathered occasionally with a veneer to coating of clay. Widely-spaced subvertical joints (closed, planar, clean to clay veneer) were also observed within the shale.

A summary of information with respect to the Queenston Formation was presented in the Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects* (March 1983), as follows.

	Uniaxial Compressive Strength (MPa)	Young’s Modulus (GPa)	Dynamic Modulus (GPa)	Poisson’s Ratio
Average	8.7	1.3	n/a	0.32
Range	7.2 to 9.6	0.5 to 2.3	n/a	0.28 to 0.35

Rock core samples were submitted for testing of unconfined compressive strength (ASTM D7012) and elastic moduli in uniaxial compression (ASTM D7012). The detailed rock laboratory testing results are appended. The test results are summarized as follows:

Borehole ID	Core ID	Depth (m)	Bulk Density (g/cm^3)	UCS (MPa)	Young’s Modulus, E (GPa)	Lithology
BH101	RC1	6.3 to 6.5	2.643	23.5	3.8	Shale
BH102	RC2	5.3 to 5.6	2.660	33.3	-	Shale

Borehole ID	Core ID	Depth (m)	Bulk Density (g/cm^3)	UCS (MPa)	Young's Modulus, E (GPa)	Lithology
BH103	RC3	7.2 to 7.5	2.656	31.1	5.3	Shale
BH104	RC4	12.3 to 12.5	2.672	44.1	10.1	Shale with Grey Banding
BH105	RC5	10.8 to 11.1	2.667	22.5	-	Shale with Grey Banding

Directly below the overburden soils, the uppermost portion of bedrock is typically weathered. The MTO (Ontario Ministry of Transportation and Communications document RR229, *Evaluation of Shales for Construction Projects)* provides a *typical weathering profile of a low durability shale* reproduced from Skempton, Davis, and Chandler, which characterizes weathered versus unweathered shale as follows:

	Zone	Description	Notes
Fully Weathered	IVb	Soil-like matrix only	indistinguishable from glacial drift deposits, slightly clayey, may be fissured
Partially Weathered	IVa	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 glacial till overburden soils directly overlying bedrock, a zone of till with fragmented shale is often observed and interpreted as either the lowest portion of the till, or as partially weathered Zone III rock. This interpretation is subjective and depends on the investigator. There is occasionally a concentration of boulders in the soil just above the bedrock that can be mistakenly identified as bedrock where rock coring is not performed. Weathering Zones III and IV are frequently not present due to glacial scouring action, which often removes these zones from the bedrock surface.

The bedrock surface as indicated on the Borehole Logs from this investigation is intended to be consistently interpreted as the surface of Zone II unless noted otherwise. Weathered and sound bedrock elevations are summarized as follows:

Borehole	Ground Surface Elevation (m)	Weathered (Zone III and Zone II) Bedrock		Unweathered/Sound (Zone I) Bedrock	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
BH101	139.5	4.3	135.2	7.6	131.9
BH102	139.8	3.4	136.4	6.7	133.1
BH103	139.3	4.0	135.3	6.9	132.4
BH104	139.7	4.0	135.7	7.8	131.9
BH105	139.4	3.7	135.7	7.6	131.8

Rock Quality Designation (RQD) is an index measurement that refers to the total length of pieces of sound core in a core run that are at least 100 mm in length, expressed as a percentage of the total length of that core run. Only natural discontinuities are used in assessing RQD. The RQD of the recovered rock cores varies between 14% and 83% in the weathered bedrock and varies between 80% and 100% in the sound bedrock.

RQD underrepresents the competency of the Queenston Formation and is not appropriate for horizontally bedded fissile shale. In this formation, the RQD is typically low due to the fissility of the shale as well as the closely spaced horizontal bedding planes. Our results are typical of this formation.

In the Queenston Formation there are near-vertical joint sets within this shale that are typically very widely spaced at over 2 m apart. There are also several faults typically referred to as “shear zones” found within the formation, which are observed as zones of rock rubble within the cores. These faults defy discovery in conventional vertical boreholes.

The jointing and crush zones in the rock are related to the state of stress in the deposit. Research in the Greater Toronto Area has revealed that the bedrock contains locked-in horizontal stresses that could be remnants of the foreshortening that occurred in the earth’s crust during continental glaciation several thousand years ago. Documented experiments have indicated that the major principal stress is of the order of 2 MPa in the upper 1 to 2 metres of the deposit where the rock is weathered and contains more fractures. Intact rock can have an internal major principal stress as high as 4 to 5 MPa. The major and minor principal stresses are horizontal and may be oriented in any direction. The empirical approach to vertical stress below the top of bedrock is to use a uniform pressure distribution below the top of bedrock elevation that is equal to the maximum earth pressure calculated for the lowest level of soil in the profile.

2.2 Groundwater

On completion of drilling, the boreholes were filled with drill fluid (from mud rotary drilling) and measuring the unstabilized groundwater level after drilling was not practical. Monitoring wells were installed in each of the boreholes, except BH102, and stabilized groundwater levels were measured in each of the monitoring wells.

The groundwater observations are shown on the Borehole Logs and are summarized as follows.

Borehole No.	Borehole depth (m)	Strata Screened	Water Level in Well, highest (m)	
			Date	Depth/Elevation
BH101	6.7	Bedrock and Sandy Silt Till	2022-05-26	4.4 / 135.1
BH102	No well installed*			
BH103	7.9	Bedrock	2022-05-26	4.5 / 134.8
BH104	9.1	Bedrock	2022-05-26	**4.5 / 135.2**
BH105	7.9	Bedrock	2022-05-26	4.6 / 134.8

*BH102 is located in an Enbridge right-of-way, no permission to install a monitoring well.

Groundwater levels fluctuate with time depending on the amount of precipitation and surface runoff, and may be influenced by known or unknown dewatering activities at nearby sites.

Grounded has measured the maximum groundwater level at Elev 135.2± m. The groundwater table is in the till deposit as well as in the underlying Queenston Formation. The glacial till deposit has a low permeability and will yield only minor seepage in the long-term. There is water within discrete fractures in the bedrock, and perched water in the earth fill.

2.3 Corrosivity and Sulphate Attack

Three (3) soil samples were submitted for corrosivity testing parameters (pH, Resistivity, Electrical Conductivity, Redox Potential, Sulphate, Sulphide and Chloride). The Certificate of Analyses is appended.

The soil samples were analysed for soluble sulphate concentration and compared to the Canadian Standard CAN3/CSA A23.1-M94 Table 3, *Additional Requirements for Concrete Subjected to Sulphate Attack*. The results are summarized as follows:

Parameter	BH102 SS1	BH103 SS6	BH104 SS4
Soluble Sulphate (SO_4) in soil sample	49 μg/g < 0.1 %	40 μg/g < 0.1 %	66 μg/g < 0.1 %
Class of Exposure	**Negligible**	**Negligible**	**Negligible**

Corrosivity parameters are also used for assessing soil corrosivity applicable to cast iron alloys, according to the 10-point soil evaluation procedure described in the American Water Work Association (AWWA) C-105 standard. The results are summarized as follows:

Parameter	AWWA C-105 Standard – Assigned Points					
	BH102 SS1		BH103 SS6		BH104 SS4	
	Result	Points	Result	Points	Result	Points
Resistivity (ohm.cm)	5830	0	3830	0	5080	0
pH	7.50	0	7.87	0	7.54	0
Redox Potential (mV)	297	0	292	0	295	0
Sulfides (%)	0.007	2	0.015	2	0.008	2
Moisture (%)	12.9	2	9.8	2	15.2	2
Corrosion protection recommended?	No		No		No	
Resistivity less than 2000 ohm.cm?	No		No		No	

The analytical results only provide an indication of the potential for corrosion. All three samples scored less than 10 points and corrosion protective measures are therefore not recommended for cast iron alloys. A more recent study by the AWWA has suggested that soil with a resistivity of less than about 2000 ohm.cm should be considered aggressive. All three samples had resistivity measurements exceeding 2000 ohm.cm.

3 Geotechnical Engineering Recommendations

Based on the factual data summarized above, we are providing the following geotechnical engineering design recommendations. Contractors must review the factual data while bidding or scoping services for this project and must provide their own opinion as to means, methods, and schedule.

This report assumes 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 Grounded should be retained to review the implications of these changes with respect to the contents of this report.

3.1 Foundation Design Parameters

The proposed development includes demolishing the existing structure and constructing a new 20-storey building with three (3) levels of underground parking. Based on the architectural set provided, the FFEs of the lowest P3 level is 129.1± m.

At this depth, the P3 level will be founded on the unweathered (sound) bedrock of the Queenston Formation. Geotechnically, the bedrock is suitable for design using spread footing foundations. We also understand that The City of Burlington may require this basement to be fully watertight, according to their new policy to disallow long-term groundwater drainage into their sewer system. The proposed structure may therefore need to be supported by a watertight raft foundation.

3.1.1 Spread Footings

Conventional spread footings made for the proposed P3 level at an FFE at 129.1± m may be made to bear on unweathered (sound) bedrock of the Queenston Formation. Conventional spread footings may be designed using a maximum factored geotechnical resistance at ULS of 5 MPa. The net geotechnical reaction at SLS is 4 MPa, for an estimated total settlement of 15 mm.

Individual spread footing foundations for footing columns must not be less than 800 mm wide and must be embedded a min. 900 mm below FFE, which applies in conjunction with the above recommended geotechnical resistance regardless of loading considerations. The geotechnical reaction at SLS refers to a settlement which, for practical purposes, is linear and non-recoverable. Differential settlement is related to column spacing, column loads, and footing sizes.

Footings stepped from one level to another should be at a slope not exceeding 1 vertical to 1 horizontal for the above bearing pressures to be applicable. There must be a minimum of 300 mm between the edge of any footing and the top of a sloped 2V:1H rock cut down to another footing.

The lowest levels of unheated underground parking structures two or more levels deep are still warmer than typical outdoor winter temperatures in the Greater Toronto Area. Interior foundations (or pile caps) with 900 mm of frost cover perform adequately, as do perimeter foundations with 600 mm of frost cover. Where foundations are next to ventilation shafts or are exposed to typical outdoor temperatures, 1.2 m of earth cover (or equivalent insulation) is required for frost protection.

The founding subgrade must be cleaned of all unacceptable materials and approved by Grounded prior to pouring concrete for the footings. Such unacceptable materials may include disturbed or caved soils, ponded water, or similar as indicated by Grounded during founding subgrade inspection. During the winter, adequate temporary frost protection for the footing bases and concrete must be provided if construction proceeds during freezing weather conditions. The bedrock surface can weather 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.

3.1.2 Raft Foundation

We understand that The City of Burlington no longer allows long-term groundwater drainage into their sewer system, which implies that basement structures must be made fully watertight. The proposed building may therefore be supported by a raft foundation, with watertight foundation

walls designed to withstand hydrostatic forces (lateral and uplift). A 20 x 36 m raft underlying the tower is considered in the bearing capacity discussion below.

Assuming a P3 FFE of 129.1± m, a raft would be founded at around Elev. 127.6± m, on unweathered (sound) Queenston Formation.

The preliminary raft design parameters are provided assuming a *uniform load* at the base of the raft. In reality, raft loads are non-uniform; they will be highest around the core and will decrease away from the core. Consequently, detailed raft design is an iterative process between the structural and geotechnical engineers. The preliminary parameters below are provided as the initial step in determining raft feasibility (a structural task).

Utilizing preliminary stiffness parameters, analysis of *uniformly* loaded 20 x 36 m raft foundation shows that a uniform SLS bearing pressure of 2,000 kPa applied at the base of the raft founded entirely on bedrock will generate around 15mm of settlement.

The modulus of subgrade reaction for design of a raft slab is a function of the size of the raft, the applied load, and whether loading is within the recompression range or the virgin range. On the basis of our preliminary stiffness parameters and the assumption of uniform raft loading, the preliminary modulus of subgrade reaction appropriate for 20 x 36 m raft design at this site uniformly on bedrock is about 140,000 kPa/m.

Settlement parameters can be improved by modelling the real non-uniform loading at the base of the raft. Detailed raft design is an iterative process between the structural and geotechnical engineers. Once a draft structural design is completed by the structural engineer, the resulting non-uniform raft pressure distribution is provided to us (typically as a contour plot). Grounded will then use finite element modelling to determine the real settlement more accurately at each point under the raft. The detailed settlement distribution and MSRs under the raft are then sent back to the structural engineer, and the structural design is modified as necessary.

The maximum factored geotechnical resistance of this raft foundation at ULS is 3,000 kPa for design purposes (assuming uniform loading).

During construction, the subgrade at founding elevation should be cut neat, inspected. Prior to pouring the raft foundation, the foundation subgrade must be cleaned of all deleterious materials such as softened, disturbed or caved materials, or standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the raft foundation base and concrete must be provided.

As the raft slab is to be fully watertight, the structure must be designed to resist uplift and lateral hydrostatic pressure on foundation walls. During construction, it will be necessary to consider the potential uplift pressure on the underside of a raft foundation due to hydrostatic forces. Positive dewatering operations during construction must begin prior to excavation and must continue until such time as the structural dead load exceeds the potential uplift forces (with suitable partial factors (LRFD) included in this assessment). A design groundwater elevation of Elev. 135.2± m is to be used.

Tiedown Anchors

If deemed necessary by the structural engineer, micropile tiedowns can be designed to resist uplift. In the bedrock below founding elevation, it is expected that conventionally grouted micropile anchors can be made such that an anchor will safely carry up to 500 kPa in tension at ULS of adhered anchor length (at a nominal diameter of 150 mm). These are preliminary parameters, subject to consultation from the design-build contractor.

These capacities are provided using resistance factors that anticipate a performance test. One or more prototype anchors must be performance-tested to 200% of the design load to demonstrate the anchor capacity and validate design assumptions. If there is no performance test to validate this capacity, anchors must be designed using a lower resistance factor. Grounded can provide these capacities on request.

3.2 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 4.1.8.4A 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 from the rational analysis of undrained shear strength (s_u) or penetration resistance (N-values) according to the OBC and National Building Code of Canada.

Below the nominal founding elevations (for spread footings or grade beams) sound bedrock was observed. Based on this information, the site designation for seismic analysis is **Class B**, per 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 same code provide the applicable acceleration- and velocity-based site coefficients.

3.3 Earth Pressure Design Parameters

At this site, the design parameters for structures subject to unbalanced earth pressures such as basement walls and retaining walls are shown in the table below.

Stratigraphic Unit	γ	φ	K_a	K_o	K_p
Compact Granular Fill Granular 'B' (OPSS.MUNI 1010)	21	32	0.31	0.47	3.25
Existing Earth Fill	19	29	0.35	0.52	2.88
Native Glacial Till or Weathered Bedrock	21	34	0.28	0.44	3.54
Sound Bedrock	26	28	n/a		

γ	=	soil bulk unit weight (kN/m^3)
φ	=	internal friction angle (degrees)
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)

These earth pressure parameters assume that grade is horizontal behind the retaining structure. If retained grade is inclined, these parameters do not apply and must be re-evaluated.

The following equation can be used to calculate the unbalanced earth pressure imposed on walls:

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

P	=	horizontal pressure (kPa) at depth h	γ	=	soil bulk unit weight (kN/m^3)
h	=	the depth at which P is calculated (m)	γ'	=	submerged soil unit weight (γ - 9.8 kN/m^3)
K	=	earth pressure coefficient	q	=	total surcharge load (kPa)
h_w	=	height of groundwater (m) above depth h			

If the wall backfill is drained such that hydrostatic pressures on the wall are effectively eliminated, this equation simplifies to:

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

If basement drainage is allowed, where walls are made directly against shoring, prefabricated composite drainage panel covering the blind side of the wall is used to provide drainage. Water from the composite drainage panel is collected and discharged through the basement wall in solid ports directly to the sumps. This is discussed in Section 3.5.

We understand that The City of Burlington may require this basement to be fully watertight, according to their new policy. In this case, the full height of the basement walls should be watertight and designed to withstand horizontal hydrostatic pressure below Elev. 135.2± m.

The possible effects of frost on retaining earth structures must be considered. In frost-susceptible soils, pressures induced by freezing pore water are basically irresistible. Insulation typically addresses this issue. Alternatively, non-frost-susceptible backfill may be specified.

Foundation resistance to sliding is proportional to the friction between the bedrock subgrade and the base of the footing. The factored geotechnical resistance to friction ($\mathbf{R_f}$) at ULS provided in the following equation:

$$R_f = \Phi N \tan \varphi$$

R_f	=	frictional resistance (kN)
Φ	=	reduction factor per Canadian Foundation Engineering Manual (CFEM) Ed. 4 (0.8)
N	=	normal load at base of footing (kN)
φ	=	internal friction angle (see table above)

3.4 Rock Swell

The earth pressure design approach for foundation walls below the top of bedrock is empirical and assumes a uniform pressure distribution below the top of bedrock elevation equal to the maximum earth pressure calculated for the lowest level of soil overtop. This approach is conventional and likely conservative, but it is practical insofar as it acknowledges that requirement of having a foundation wall of a consistent width at the lower levels.

However, this approach does not recognize the potential for pressures on the basement wall due to time-dependent rock swell that results when locked in horizontal stresses are released. For structures deeper than 2 m below the top of sound rock, rock swell must also be considered. The simplest approach to dealing with rock swell is scheduling. If there is a 120-day gap between rock excavation and construction of the permanent structure that will restrain the rock, experience on similar structures indicates the rock will de-stress and swell, and no significant stresses are imposed on the structural wall. This requirement typically only impacts the lowest 2 basement levels in bedrock, acknowledging the 120-day window.

If the construction schedule does not allow for the 120-day gap described above, mitigation measures will be required. For structures subjected unbalanced rock swell pressure (i.e. lowest exterior foundation walls, sumps, elevators, other features cast directly against the rock face), rock squeeze effects can be addressed by providing a crushable layer between the rock and the concrete, such as 50 mm thick Ethafoam 220 Polyethylene Foam planks. The subject walls are typically designed for the 50% compressive strength resistance of the foam. At 50% compression, a 220 Ethafoam 220 Polyethylene Foam plank provides 124 kPa of resistance. At 10% compression (which allows for concrete placement), this material provides 50 kPa of resistance.

Deeper protrusions (sumps, elevator pits, etc.) can be over-excavated as they are not typically constrained by the property lines or adjacent footings. In this case the rock can be horizontally over-excavated by a minimum 600 mm on all sides. Precast pits and sumps are then placed and backfilled with 19 mm clear stone (OPSS.MUNI 1004). The clear stone backfill then accommodates the rock swell.

Rock squeeze effects are not relevant to foundation excavations as the earth pressures exerted on foundation elements are balanced, and concrete is strong enough to resist the swell pressure and render it null.

3.5 Slab on Grade Design Parameters

The slab-on-grade parameters provided here apply to a conventional slab on grade and drained basement approach only. If a fully watertight raft foundation approach is adopted (with no permanent drainage system), design parameters are provided in Section 3.1.2.

At the proposed lowest (P3) basement slab FFE of 129.1± m, the unweathered (sound) bedrock of the Queenston Formation will provide adequate subgrade for support of a conventional slab

on grade. The modulus of subgrade reaction appropriate for design of the slab resting on an aggregate drainage layer base overlying bedrock is 80,000 kPa/m.

The slab on grade must be provided with a drainage layer and capillary moisture break, which is achieved by forming the slab on a minimum 300 mm thick layer of 19 mm clear stone (OPSS 1004) vibrated to a dense state.

The use of excavated bedrock spoil to restore subgrade elevations is to be specifically prohibited. This bedrock spoil cannot be adequately compacted to provide support for the slab on grade and is not to be reused below any settlement sensitive areas.

If this basement structure is made as a conventional drained structure, a permanent drainage system including subfloor drains is required (see Section 3.5). Subfloor drains are typically installed in trenches below the capillary moisture break drainage layer per the typical detail appended. If trenches are to be avoided, the subfloor drainage system can be incorporated into the capillary moisture break and drainage layer. The subfloor drains are then laid directly on the flat subgrade and backfilled with a minimum 300 mm thick layer of 19 mm clear stone (OPSS 1004) vibrated to a dense state. Any solid collection pipes must be sloped so that they positively discharge to the sumps.

3.6 Long-Term Groundwater and Seepage Control

The requirement for a permanent basement drainage system depends on whether a fully watertight approach is adopted for this site. The City of Burlington may require this basement to be fully watertight, according to their new policy.

3.6.1 Drained Basement

For a conventional drained basement approach, perimeter and subfloor drainage systems are required for the underground structure. Subfloor drainage collects and removes the seepage that infiltrates under the floor. Perimeter drainage collects and removes seepage that infiltrates at the foundation walls. The exterior faces of foundation walls should be provided with a layer of waterproofing to protect interior finishes.

Subfloor drainage pipes are to be spaced at an average 6 m (measured on-centres). If subdrain elevation conflicts with top of footing elevation, footings should be lowered as necessary.

To limit seepage to the extent practicable, exterior grades adjacent to foundation walls should be sloped at a minimum 2 percent gradient away from the wall for 1.2 m minimum.

The walls of the substructure are to be fully drained to eliminate hydrostatic pressure. Where drained basement walls are made directly against shoring, prefabricated composite drainage panel covering the blind side of the wall is used to provide drainage. Seepage from the composite drainage panel is collected and discharged through the basement wall in solid ports directly to

the sumps. A layer of waterproofing placed between the drain core product and the basement wall should be considered to protect interior finishes from moisture. Typical details are appended.

The perimeter and subfloor drainage systems are critical structural elements since they eliminate hydrostatic pressure from acting on the basement walls and floor slab. The sumps that ensure the performance of these systems must have a duplexed pump arrangement providing 100% redundancy, and they must be on emergency power. The sumps should be sized by the mechanical engineer to adequately accommodate the estimated volume of water seepage.

3.6.2 Watertight Basement

If required by the City of Burlington, the structure can be fully watertight and designed to withstand hydrostatic pressures, with no permanent drainage system. The full height of the basement walls should be watertight (no drainage) and designed to withstand hydrostatic pressure (horizontal and uplift) using a static groundwater table at Elev. 135.2± m.

4 Considerations for Construction

4.1 Excavations

Excavations must be carried out in accordance with the *Occupational Health and Safety Act – Regulation 213/91 – Construction Projects (Part III - Excavations, Section 222 through 242).* These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. For practical purposes:

- The earth fill is a Type 3 soil
- The till is a Type 3 soil

In accordance with the regulation's requirements, the soil must be suitably sloped and/or braced where workers must enter a trench or excavation deeper than 1.2 m. Safe excavation slopes (of no more than 3 m in height) by soil type are stipulated 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 Sections 235 through 238 and 241 of the Act and Regulations and include provisions for timbering, shoring and moveable trench boxes. Any excavation slopes greater than 3 m in height should be checked by Grounded for global stability issues.

Bedrock is not considered a soil under the Act. Vertical excavations made in sound bedrock are generally self-supporting provided the rock bedding is horizontally oriented. If deemed necessary, rock bolts can be used to anchor a layer of protective mesh that will protect workers from loose rock spalling from the face of excavation. The rock face must be inspected by Grounded to determine that no other support system is required to prevent the spalling of loose rock, and to confirm that all loose spall material at risk of falling upon a worker is removed (Section 233 of the above noted regulations).

Larger obstructions (e.g. buried concrete debris, other obstructions) not directly observed in the boreholes are likely present in the earth fill. Similarly, larger inclusions (e.g. cobbles and boulders) may be encountered in the native soils. The size and distribution of these obstructions cannot be predicted with boreholes, as the split spoon sampler is not large enough to capture particles of this size. Provision must be made in excavation contracts to allocate risks associated with the time spent and equipment utilized to remove or penetrate such obstructions when encountered.

Queenston Formation bedrock is a rippable rock that can be removed with conventional excavation equipment once it has been broken by ripper tooth or hoe ram. Creating detailed excavation shapes for foundations etc. is normally accomplished by hoe ram. The removal of rock from a vertical face without over-excavation, which can happen inadvertently by dislodging additional rock, is largely dependent on machine operator skill. If excavation faces must be made neat (such as beside an existing footing), a line of excavation can be provided by line drilling the rock a series of closely-spaced vertical holes (100 mm diameter, spaced at 300 mm on centre) to provide a preferential vertical break path for the excavation face.

Queenston Formation bedrock contains beds of harder limestone. When excavating this bedrock, it should be expected that these beds will be encountered. Hard layers of limestone interbedded within the shale are normally broken with hoe mounted hydraulic rams before excavation.

Limestone beds may also be found to straddle the founding elevation, in which case the entire thickness of the hard limestone layer must be removed to expose founding subgrade as it is not possible to remove part of one of these layers. This will in turn 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.

4.2 Short-Term Groundwater Control

Considerations pertaining to groundwater discharge quantities and quality are discussed in the Hydrogeological Report (by another consultant).

The groundwater table at Elev. 135.2± m is below the bulk excavation level for P3. Within the zone of excavation, the boreholes were generally dry with no seepage. There may be perched water in the fill or fracture zones in the bedrock. On this basis, it is expected that groundwater, if encountered, will be of limited extent. Seepage from these zones may be allowed to drain into the

excavation and then controlled by a conventional sump pump arrangement. The volume of seepage anticipated in open excavations is limited to the extent that temporary pumping from the excavations is expected to sufficiently control groundwater seepage. Regardless, delays in excavation will occur as the seepage is controlled and these delays should be anticipated in the construction schedule.

A watertight basement may be required. During construction, it will be necessary to consider the potential uplift pressure on the underside of a raft foundation due to hydrostatic forces. Positive dewatering operations during construction must begin prior to excavation and must continue until such time as the structural dead load exceeds the potential uplift forces (with suitable partial factors (LRFD) included in this assessment). A design groundwater elevation of Elev. 135.2± m is to be used in this assessment.

The City of Burlington may require a Discharge Agreement in the short term, if any water is to be discharged to the storm or sanitary sewers during construction.

4.3 Earth-Retention Shoring Systems

The site is immediately bounded by Appleby Line to the west, a utility corridor to the north, and existing buildings to the east and south. No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided. Underpinning guidelines are appended.

Continuous interlocking caisson wall shoring is to be used where the excavation must be constructed as a rigid shoring system. Caisson wall shoring preserves the support capabilities and integrity of the soil beneath existing foundations of adjacent buildings, in a state akin to the at-rest condition. Otherwise, excavations can be supported using conventional soldier pile and lagging walls.

4.3.1 Lateral Earth Pressure Distribution

If the shoring is supported with a single level of earth anchor or bracing, a triangular earth pressure distribution like that used for the basement wall design is appropriate.

Where multiple rows of lateral supports are used to support the shoring walls, research has shown that a distributed pressure diagram more realistically approximates the earth pressure on a shoring system of this type, when restrained by pre-tensioned anchors. A multi-level supported shoring system can be designed based on an earth pressure distribution with a maximum pressure defined by:

$$\boldsymbol{P = 0.8\,K[\gamma H + q] + \gamma_w h_w}$$

P = maximum horizontal pressure (kPa)
K = earth pressure coefficient (see Section 3.3)
H = total depth of the excavation (m)
h_w = height of groundwater (m) above the base of excavation

γ = soil bulk unit weight (kN/m3)
q = total surcharge loading (kPa)

Where shoring walls are drained to effectively eliminate hydrostatic pressure on the shoring system (e.g. pile and lagging walls), h_w is equal to zero. For the design of impermeable shoring, a design groundwater table at Elev. 135.2± m must be accounted for.

In cohesive soils, the lateral earth pressure distribution is trapezoidal, uniformly increasing from zero to the maximum pressure defined in the equation above over the top and bottom quarter (H/4) of the shoring.

Where the excavation penetrates the bedrock, the rock excavation is nominally self-supporting in a vertical face, provided the rock bedding is horizontally oriented. The requirement for extending lagging into partially weathered rock depends on the quality of the excavation cut and the degree of weathering.

4.3.2 Soldier Pile Toe Embedment

Soldier pile toes will be made in the bedrock. Soldier pile toes resist horizontal movement due to the passive earth pressure acting on the toe below the base of excavation. The maximum factored vertical geotechnical resistance at ULS for the design of a pile embedded in the unweathered (sound) bedrock is 10 MPa. The maximum factored lateral geotechnical resistance at ULS of the undisturbed rock is 1 MPa.

Exposed bedrock of the Queenston Formation deteriorates with time. Within 12 months of exposure, excavation faces made within this bedrock flake and recede as much as 300 mm, generally in the form of coin-size shale particles dropping from the face on a constant basis. The deteriorated rock loses internal integrity and bearing capability. Solider piles for the shoring system are typically advanced at least 1 meter below the base of the excavation to accommodate this weathering and still ensure that the required lateral and vertical bearing resistances can be utilized.

4.3.3 Lateral Bracing Elements

The shoring system at this site may require lateral bracing. In this case, if feasible, the shoring system should be supported by pre-stressed soil anchors (tiebacks) extending into the subgrade of the adjacent properties. To limit the movement of the shoring system as much as is practically possible, tiebacks are installed and stressed as excavation proceeds. The use of tiebacks through adjacent properties requires the consent (through encroachment agreements) of the adjacent property owners.

Conventional rock anchors made in Queenston Formation bedrock can be designed using a working adhesion of 500 kPa.

At least one prototype anchor per tieback level must be performance-tested to 200% of the design load to demonstrate the anchor capacity and validate design assumptions. Given the potential variability in soil conditions or installation quality, all production anchors must also be proof-tested to 133% of the design load.

The Queenston Formation bedrock below the proposed P3 FFE (Elev. 129.1 m) is suitable for the placement of raker foundations. Raker footings established on bedrock at an inclination of 45 degrees can be designed using a maximum factored geotechnical resistance at ULS of 2,000 kPa.

4.4 Site Work

To better protect wet undisturbed subgrade, excavations exposing wet soils must be cut neat, inspected, and then immediately protected with a skim coat of concrete (i.e. a mud mat). Wet sands are susceptible to degradation and disturbance due to even mild site work, frost, weather, or a combination thereof.

The effects of work on site can greatly impact soil integrity. Care must be taken to prevent this damage. Site work carried out during periods of inclement weather may result in the subgrade becoming disturbed, unless a granular working mat is placed to preserve the subgrade soils in their undisturbed condition. Subgrade preparation activities should not be conducted in wet weather and the project must be scheduled accordingly.

If site work causes disturbance to the subgrade, removal of the disturbed soils and the use of granular fill material for site restoration or underfloor fill will be required at additional cost to the project.

It is construction activity itself that often imparts the most severe loading conditions on the subgrade. Special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during placement of the granular base and other work may be required, especially if construction is carried out during unfavourable weather.

Adequate temporary frost protection for the founding subgrade must be provided if construction proceeds in freezing weather conditions. The subgrade at this site is susceptible to frost damage. The slab on grade should not be placed on frozen subgrade, to prevent settlement of the slab as the subgrade thaws. Areas of frozen subgrade should be removed during subgrade preparation. Depending on the project context, consideration should be given to frost effects (heaving, softening, etc.) on exposed subgrade surfaces.

The exposed Queenston 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. If bedrock is to be exposed for prolonged periods of time, it is recommended that a skim coat of concrete be used to protect the surface of bedrock from slaking and other degradation resulting from weathering.

4.5 Engineering Review

By issuing this report, Grounded Engineering has assumed the role of Geotechnical Engineer of Record for this site. Grounded should be retained to review the structural engineering drawings prior to issue or construction to ensure that the recommendations in this report have been appropriately implemented.

All foundation installations must be reviewed in the field by Grounded, the Geotechnical Engineer of Record, as they are constructed. The on-site review of the condition of the founding subgrade as the foundations are constructed is as much a part of the geotechnical engineering design function as the design itself; it is also required by Section 4.2.2.2 of the Ontario Building Code. If Grounded is not retained to carry out foundation engineering field review during construction, then Grounded accepts no responsibility for the performance or non-performance of the foundations, even if they are constructed in general conformance with the engineering design advice contained in this report.

The long-term performance of a slab on grade is highly dependent upon the subgrade support and drainage conditions. Strict procedures must be maintained during construction to maintain the integrity of the subgrade to the extent 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 Grounded at the time of construction to confirm material quality, thickness, and to ensure adequate compaction.

A visual pre-construction survey of adjacent lands and buildings is recommended to be completed prior to the start of any construction. This documents the baseline condition and can prevent unwarranted damage claims. Any shoring system, regardless of the execution and design, has the potential for movement. Small changes in stress or soil volume can cause cracking in adjacent buildings.

5 Limitations and Restrictions

5.1 Investigation Procedures

The geotechnical engineering analysis and advice provided here are based on the factual borehole information observed and recorded by Grounded. The investigation methodology and engineering analysis methods used to carry out this scope of work are consistent with conventional standard practice by Grounded as well as other geotechnical consultants, working under similar conditions and constraints (time, financial and physical).

Borehole drilling services were provided to Grounded by a specialist professional contractor. The drilling was observed and recorded by Grounded's field supervisor on a full-time basis. Drilling

was conducted using conventional drilling rigs equipped with hollow stem augers and mud rotary drilling equipment. Rock coring will be carried out with HQ size diamond bit core drilling barrels. As drilling proceeded, groundwater observations were made in the boreholes. Based on examination of recovered borehole samples, our field supervisor made a record of borehole and drilling observations. The field samples were secured in air-tight clean jars and bags and taken to the Grounded soil laboratory where they were each logged and reviewed by the geotechnical engineering team and the senior reviewer.

The Split-Barrel Method technique (ASTM D1586) was used to obtain the soils samples. The sampling was conducted at conventional intervals and not continuously. As such, stratigraphic interpolation between samples is required and stratigraphic boundary lines do not represent exact depths of geological change. They should be taken as gradual transition zones between soil or rock types.

A carefully conducted, fully comprehensive investigation and sampling scope of work carried out under the most stringent level of oversight may still fail to detect certain ground conditions. As such, users of this report must be aware of the risks inherent in using engineered field investigations to observe and record subsurface conditions. As a necessary requirement of working with discrete test locations, Grounded has assumed that the conditions between test locations are the same as the test locations themselves, for the purposes of providing geotechnical engineering advice.

It is not possible to design a field investigation with enough test locations that would provide complete subsurface information, nor is it possible to provide geotechnical engineering advice that completely identifies or quantifies every element that could affect construction, scheduling, or tendering. Contractors undertaking work based on this report (in whole or in part) must make their own determination of how they may be affected by the subsurface conditions, based on their own analysis of the factual information provided and based on their own means and methods. Contractors using this report must be aware of the risks implicit in using factual information at discrete test locations to infer subsurface conditions across the site and are directed to conduct their own investigations as needed.

5.2 Site and Scope Changes

Natural occurrences, the passage of time, local construction, and other human activity all have the potential to directly or indirectly alter the subsurface conditions at or near the project site. Contractual obligations related to groundwater or stormwater control, disturbed soils, frost protection, etc. must be considered with attention and care as they relate this potential site alteration.

The geotechnical engineering advice provided in this report is based on the factual observations made from the site investigations as reported. It is intended for use by the owner and their retained design team. If there are changes to the features of the development or to the scope, the interpreted subsurface information, geotechnical engineering design parameters, advice, and

discussion on construction considerations may not be relevant or complete for the project. Grounded should be retained to review the implications of such changes with respect to the contents of this report.

5.3 Report Use

The authorized users of this report are Latch Developments and their design team, for whom this report has been prepared. Grounded Engineering Inc. maintains the copyright and ownership of this document. Reproduction of this report in any format or medium requires explicit prior authorization from Grounded Engineering Inc.

The City of Burlington may also make use of and rely upon this report, subject to the limitations as stated.

6 Closure

If the design team has any questions regarding the discussion and advice provided, please do not hesitate to have them contact our office. We trust that this report meets your requirements at present.

For and on behalf of our team,

GROUNDED ENGINEERING





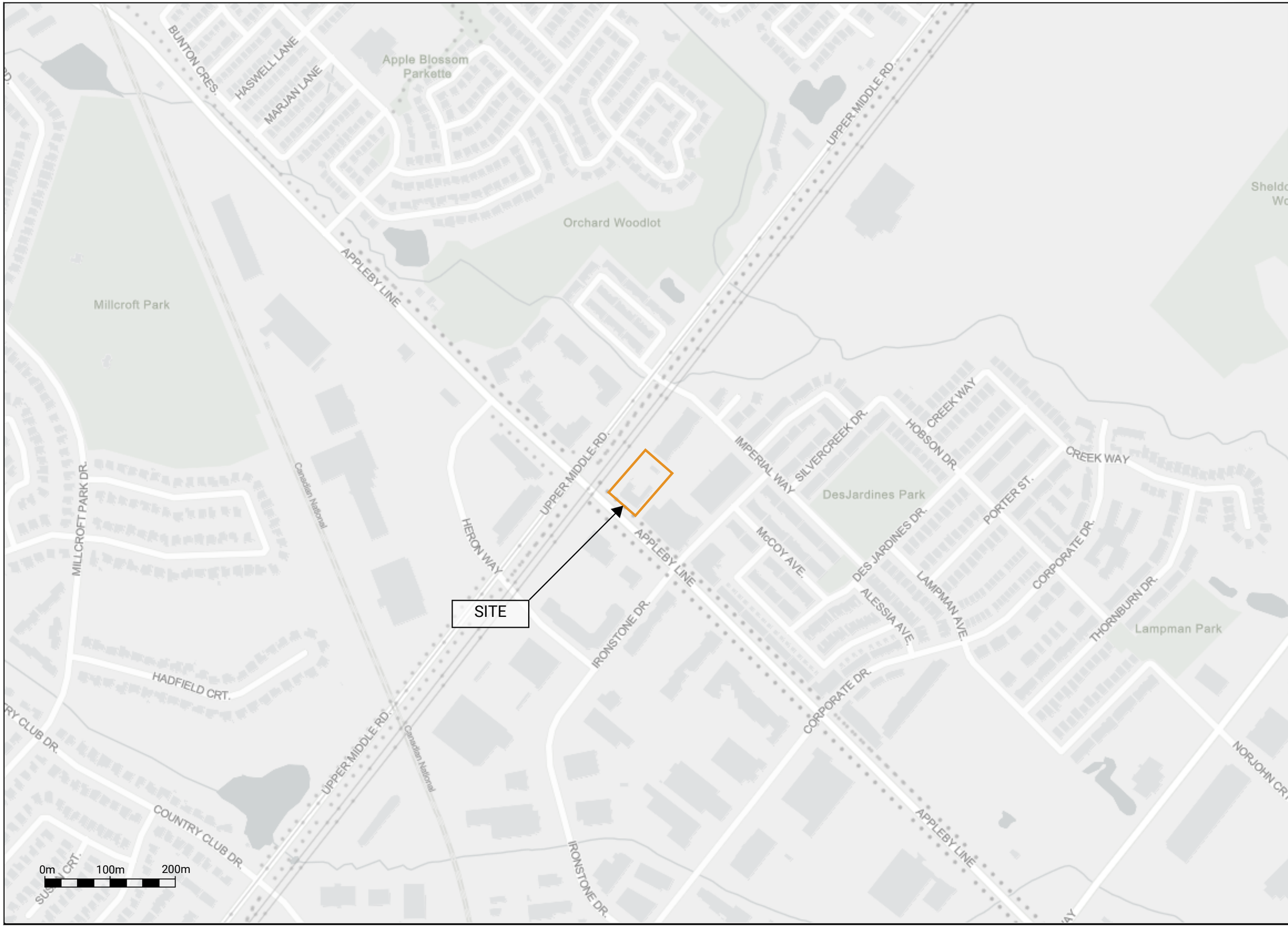
Kim Falkenstein, M.A.Sc., P.Eng.

Michael Diez de Aux, M.A.Sc., P.Geo., P.Eng.
Associate

Jason Crowder, Ph.D., P.Eng.

FIGURES





GROUNDED ENGINEERING
12 Banigan Drive, Toronto, Ont., M4H 1E9
www.groundedeng.ca

LEGEND

APPROXIMATE PROPERTY BOUNDARY

Note

Reference

ArcGIS My Map 2022

Project

1989 APPLEBY LINE, BURLINGTON, ONTARIO

Figure Title

SITE LOCATION PLAN

North

Date

MAY 2022

Scale

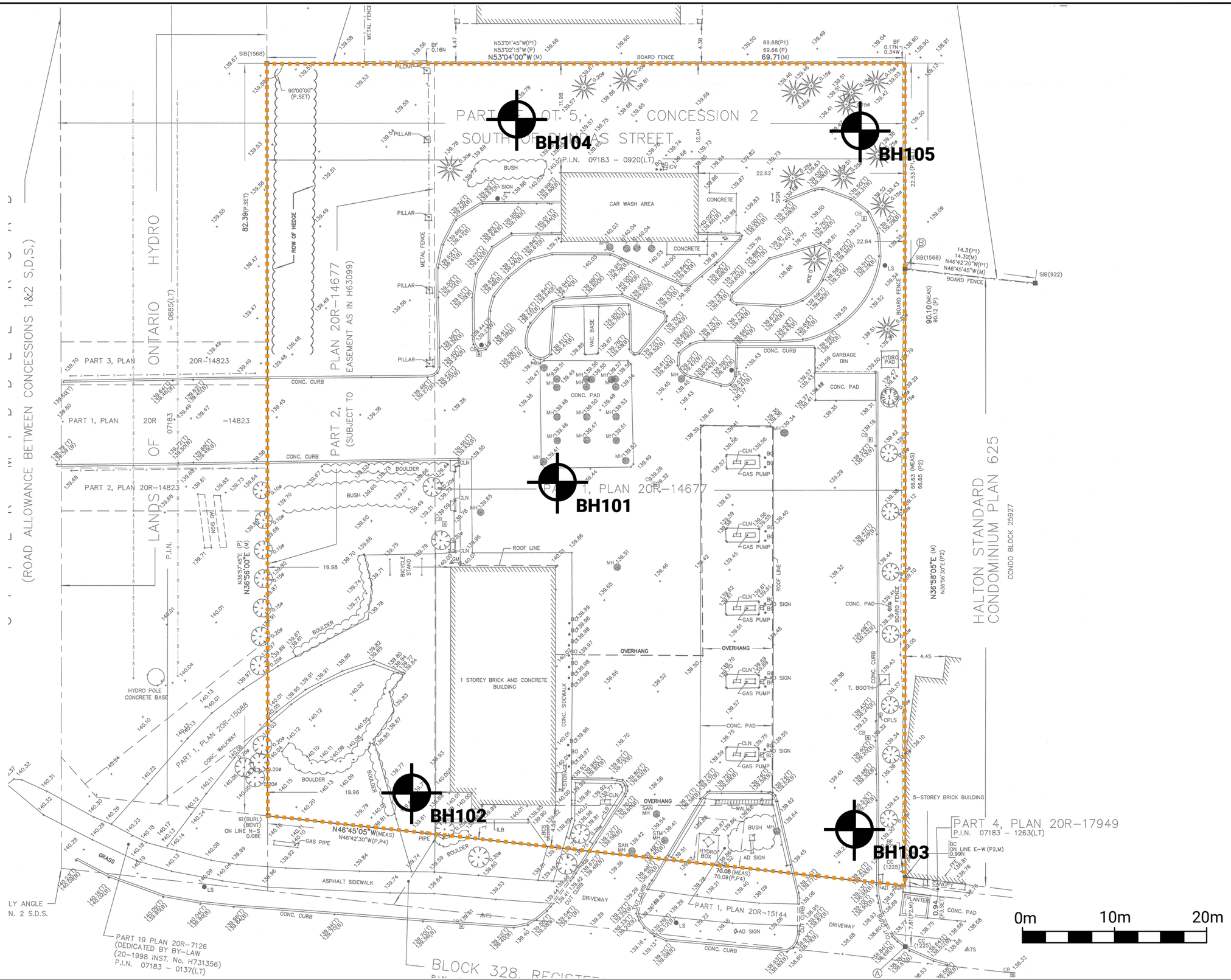
AS INDICATED

Job No

22-063

Figure No

FIGURE 1



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LEGEND

- APPROXIMATE PROPERTY BOUNDARY
- MONITORING WELL/BOREHOLE BY GROUNDED

Note

Reference

Survey Drawing. Project No. 3504-0.
Prepared by R. Avis Surveying Inc. Dated September 27, 2021.

Project

1989 APPLEBY LINE, BURLINGTON, ONTARIO

Figure Title

BOREHOLE LOCATION PLAN

North

TRUE
PROJECT

Date

MAY 2022

Scale

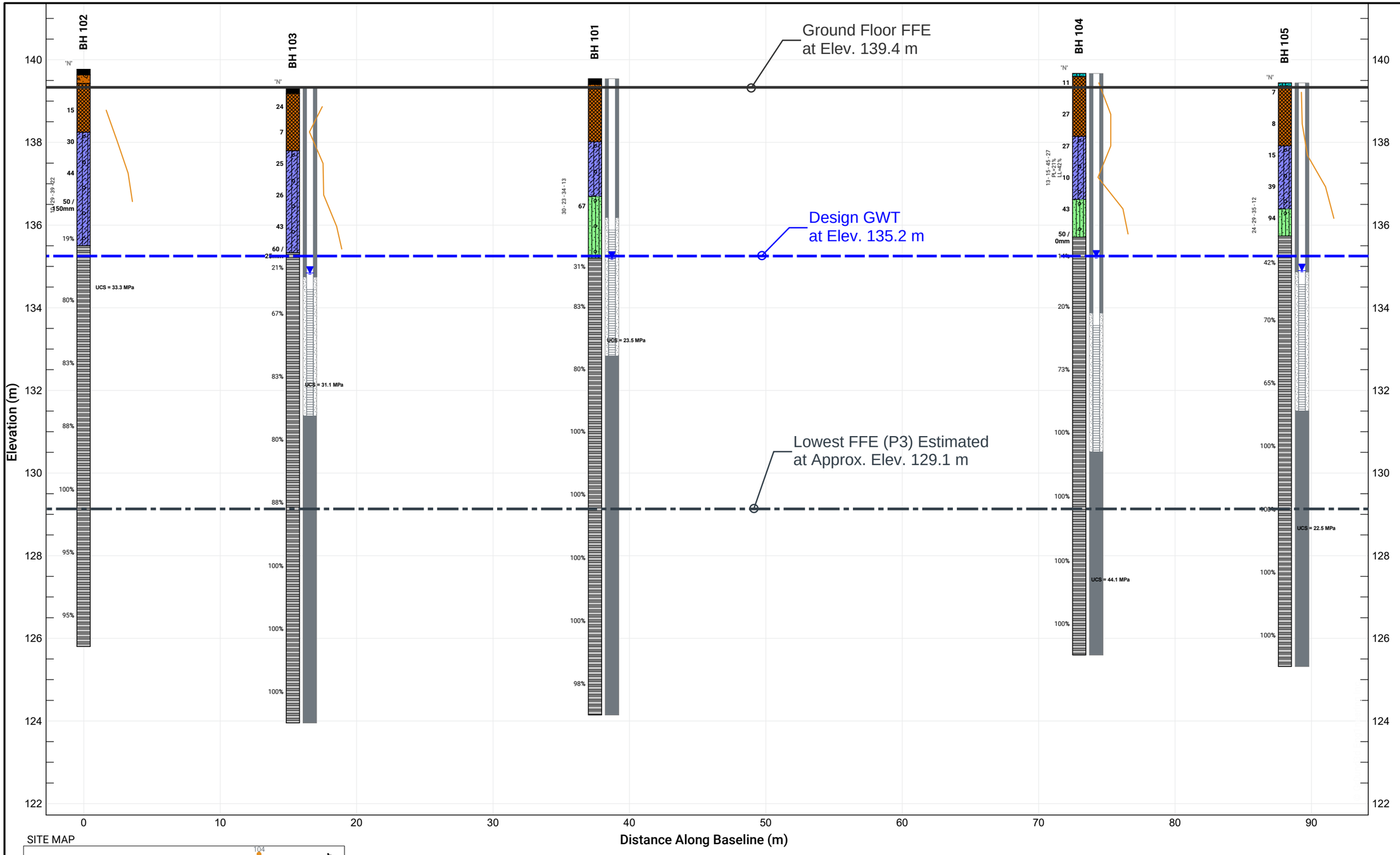
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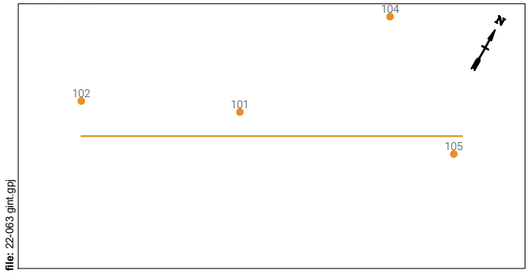
22-063

Figure No

FIGURE 2



SITE MAP



file: 22-063 grnt.gpj

BOREHOLE STRATIGRAPHY LEGEND

- Asphalt
- Aggregate
- Fill
- Clayey Silt Till
- Sandy Silt Till
- Bedrock (cored)
- Topsoil

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LEGEND

- FILL
- GRAVELS (gravel to gravelly sand)
- SILT TO SAND (not till)
- COHESIONLESS TILLS
- COHESIVE SOILS (clayey silt to clay, incl. tills)
- DISTURBED/REWORKED/ORGANIC

Project
1989 APPLEBY LINE
BURLINGTON, ONTARIO

Figure Title
SUBSURFACE PROFILE

Date
MAY 2022

Scale
AS INDICATED

Job No
22-063

Figure No
FIGURE 3

APPENDIX A



BOREHOLE LOG TERMINOLOGY

GROUNDED ENGINEERING

SAMPLING/TESTING METHODS

SS: split spoon sample

AS: auger sample

GS: grab sample

FV: shear vane

DP: direct push

PMT: pressuremeter test

ST: shelby tube

CORE: soil coring

RUN: rock coring

SYMBOLS & ABBREVIATIONS

MC: moisture content

LL: liquid limit

PL: plastic limit

PI: plasticity index

γ: soil unit weight (bulk)

G_s: specific gravity

S_u: undrained shear strength

▽ unstabilized water level

▼ 1st water level measurement

▼ 2nd water level measurement most recent

▼ water level measurement

ENVIRONMENTAL SAMPLES

M&I: metals and inorganic parameters

PAH: polycyclic aromatic hydrocarbon

PCB: polychlorinated biphenyl

VOC: volatile organic compound

PHC: petroleum hydrocarbon

BTEX: benzene, toluene, ethylbenzene and xylene

PPM: parts per million

FIELD MOISTURE (based on tactile inspection)

DRY: no observable pore water

MOIST: inferred pore water, not observable (i.e. grey, cool, etc.)

WET: visible pore water

COHESIONLESS

Relative Density	N-Value
Very Loose	<4
Loose	4 - 10
Compact	10 - 30
Dense	30 - 50
Very Dense	>50

COHESIVE

Consistency	N-Value	Su (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	% by weight
trace silt	<10
some silt	10 - 20
silt***y***	20 - 35
sand ***and*** silt	>35

ASTM STANDARDS

ASTM D1586 Standard Penetration Test (SPT)
Driving a 51 mm O.D. split-barrel sampler ("split spoon") into soil with a 63.5 kg weight free falling 760 mm. The blows required to drive the split spoon 300 mm ("bpf") after an initial penetration of 150 mm is referred to as the N-Value.

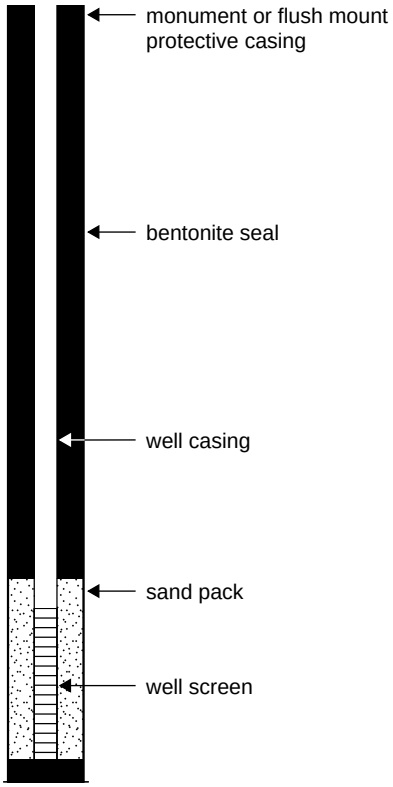
ASTM D3441 Cone Penetration Test (CPT)
Pushing an internal still rod with a outer hollow rod ("sleeve") tipped with a cone with an apex angle of 60° and a cross-sectional area of 1000 mm^2 into soil. The resistance is measured in the sleeve and at the tip to determine the skin friction and the tip resistance.

ASTM D2573 Field Vane Test (FVT)
Pushing a four blade vane into soil and rotating it from the surface to determine the torque required to shear a cylindrical surface with the vane. The torque is converted to the shear strength of the soil using a limit equilibrium analysis.

ASTM D1587 Shelby Tubes (ST)
Pushing a thin-walled metal tube into the in-situ soil at the bottom of a borehole, removing the tube and sealing the ends to prevent soil movement or changes in moisture content for the purposes of extracting a relatively undisturbed sample.

ASTM D4719 Pressuremeter Test (PMT)
Place an inflatable cylindrical probe into a pre-drilled hole and expanding it while measuring the change in volume and pressure in the probe. It is inflated under either equal pressure increments or equal volume increments. This provides the stress-strain response of the soil.

WELL LEGEND



ROCK CORE TERMINOLOGY (MTO SHALE)

TCR **Total Core Recovery** the total length of recovery (soil or rock) per run, as a percentage of the drilled length
SCR **Solid Core Recovery** the total length of sound full-diameter rock core pieces per run, as a percentage of the drilled length
RQD **Rock Quality Designation** the sum of all pieces of sound rock core in a run which are 10 cm or greater in length, as a percentage of the drilled length

Natural Fracture Frequency (typically per 0.3 m) The number of natural discontinuities (joints, faults, etc.) which are present per 0.3m. Ignores mechanical or drill-induced breaks, and closed discontinuities (e.g. bedding planes).

LOGGING DISCONTINUITIES

Discontinuity Type

BP bedding parting
CL cleavage
CS crushed seam
FZ fracture zone
MB mechanical break
IS infilled seam
JT Joint
SS shear surface
SZ shear zone
VN vein
VO void

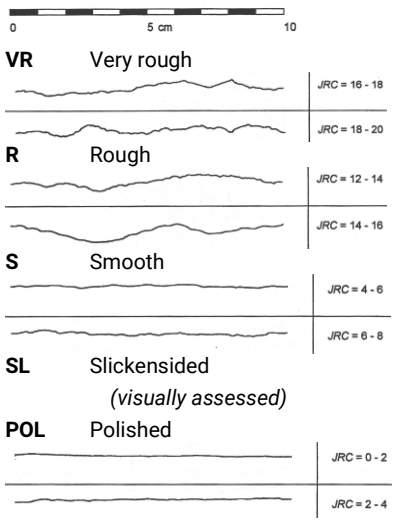
Coating

CN Clean
SN Stained
OX Oxidized
VN Veneer
CT Coating (>1 mm)

Dip Inclination

H	horizontal/flat	0 - 20°
D	dipping	20 - 50°
SV	sub-vertical	50 - 90°
V	vertical	90±°

Roughness (Barton et al.)



Spacing in Discontinuity Sets (*ISRM 1981*)

VC	very close	< 60 mm
C	close	60 – 200 mm
M	mod. close	0.2 to 0.6 m
W	wide	0.6 to 2 m
VW	very wide	> 2 m

Aperture Size

T	closed / tight	< 0.5 mm
GA	gapped	0.5 to 10 mm
OP	open	> 10 mm

Planarity

PR	Planar
UN	Undulating
ST	Stepped
IR	Irregular
DIS	Discontinuous
CU	Curved

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; ISRM 1981b*)

Grade		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 (*Q. J. Eng. Geology, Vol 3, 1970*)

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

GROUNDED ENGINEERING

Date Started : Apr 4, 2022
Position : E: 598066, N: 4805252 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 101

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

drill method : CME 75

elev depth (m)	description	graphic log	number	type	SPT N-value
139.5	**GROUND SURFACE**				
	140mm **ASPHALT**		1	GS	
	100mm **AGGREGATE**				
	FILL, silty sand, some gravel, trace rock fragments, trace asphalt, brown and dark brown, moist				
138.0 / 1.5	**CLAYEY SILT**, some sand, trace gravel, dark brown, moist (GLACIAL TILL)		2	GS	
136.7 / 2.8	**SANDY SILT**, some gravel, some clay, trace rock fragments, very dense, reddish brown, moist (GLACIAL TILL)		3	SS	67
135.2 / 4.3	**QUEENSTON FORMATION** (See rock core log for details)		1	RUN	
			2	RUN	
			3	RUN	
			4	RUN	
			5	RUN	
			6	RUN	
			7	RUN	
			8	RUN	
124.1 / 15.4					

hydro-vac OD=300 mm

hollow stem augers (mech) OD=175 mm

Rock coring (HQ) OD=96 mm

depth scale (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

well details

elevation (m): 139 138 137 136 135 134 133 132 131 130 129 128 127 126 125

undrained shear strength (kPa)
○ unconfined + field vane
● pocket penetrometer ■ Lab Vane
40 80 120 160

SPT N-values (bpf)
× dynamic cone
10 20 30 40

headspace vapour (ppm)
× hexane □ isobutylene
⊠ methane
100 200 300

moisture / plasticity
PL MC LL
10 20 30

unstabilized water level

lab data and comments

grain size distribution (%) (MIT)
GR SA SI CL

30 23 34 13

3.5m: spoon bouncing, auger grinding

...at 7.6 m, top of sound bedrock

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
May 26, 2022	4.4	135.1

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 4, 2022
Position : E: 598066, N: 4805252 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 101

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

Rock coring started at 4.3m below grade

QUEENSTON FORMATION
Shale, reddish brown, thinly bedded to medium bedded, weak, grey-green banding; joints are horizontal, closed to gapped, clean to sandy / silty filling

... at 7.6 m (Elev. 131.9 m), transition to sound rock

Run	elev depth (m)	recovery
	135.2	
R1	4.3	TCR = **100%** SCR = **44%** RQD = **31%**
	4.7	
R2		TCR = **100%** SCR = **95%** RQD = **83%**
	133.2	
R3	6.3	TCR = **100%** SCR = **91%** RQD = **80%**
	131.7	
R4	7.8	TCR = **100%** SCR = **100%** RQD = **100%**
	130.2	
R5	9.3	TCR = **100%** SCR = **100%** RQD = **100%**
	128.7	
R6	10.8	TCR = **100%** SCR = **100%** RQD = **100%**
	127.2	
R7	12.3	TCR = **100%** SCR = **100%** RQD = **100%**
	125.6	
R8	13.9	TCR = **100%** SCR = **98%** RQD = **98%**
	124.1	
	15.4m	

END OF COREHOLE

natural fracture frequency: 1+RZ, 2+SM, 2+SM, 1, 0, 1+SM, 0, 1, 2, 4, 1+SM, 2, 0, 1, 0, 0, 1, 0, 0, 0, 0, 1, 0, 0, 0, 0, 0, 0, 0, 0, 0, 0, 2, 1, 0

laboratory testing: **El. 133.2m**: UCS = 23.5 MPa, E = 3.80 GPa, γ = 25.9 kN/m^3

notes and comments

4.3 / 135.2 - 4.6 / 135.0m: Rubblized
5.1 / 134.5m: Clay seam (10 mm)
5.3 / 134.2m: Clay seam (25 mm)
6.1 / 133.4m: Clay seam (25 mm)
7.4 / 132.1 - 7.5 / 132.0m: Weathered zone
7.5 / 132.0m: Clay seam (50 mm)
14.7 / 124.9m: JT D PR GA VN

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 6, 2022
Position : E: 598032, N: 4805236 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 102

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

drill method : CME 55

elev / depth (m)	description	graphic log	samples: number	samples: type	samples: SPT N-value
139.8	**GROUND SURFACE**				
139.5 / 0.3	140mm **ASPHALT**				
	200mm **AGGREGATE**				
	FILL, clayey silt, some sand, trace gravel, trace brick fragments, organic odour, stiff, brown and dark brown, moist		1	SS	15
138.3 / 1.5	**CLAYEY SILT**, some sand, trace gravel, hard, brown and grey, moist (GLACIAL TILL)		2	SS	30
	...at 2.3 m, sandy		3	SS	44
	...at 3.3 m, some gravel, some clay, some rock fragments (shale fragments), very dense, reddish brown		4A / 4B	SS	50 / 150mm
135.5 / 4.3	**QUEENSTON FORMATION** (See rock core log for details)		1	RUN	
			2	RUN	
			3	RUN	
			4	RUN	
			5	RUN	
			6	RUN	
125.8 / 14.0			7	RUN	

hollow stem augers (med) OD=175 mm

Rock coring (HQ) OD=96 mm

depth scale (m): 0 1 2 3 4 5 6 7 8 9 10 11 12 13

well details

elevation (m): 139 138 137 136 135 134 133 132 131 130 129 128 127 126

undrained shear strength (kPa)
○ unconfined + field vane
● pocket penetrometer ■ Lab Vane
40 80 120 160

SPT N-values (bpf)
× dynamic cone
10 20 30 40

headspace vapour (ppm)
× hexane □ isobutylene
⊠ methane
100 200 300

moisture / plasticity
PL MC LL
10 20 30

unstabilized water level

lab data and comments

grain size distribution (%) (MIT)
GR SA SI CL

10 29 39 22

...at 6.7 m, top of sound bedrock

END OF BOREHOLE

Contained drill water upon completion of drilling. Unstabilized water level and cave not measured.

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 6, 2022
Position : E: 598032, N: 4805236 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 102

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

depth (m)	stratigraphy	Run elev depth (m)	recovery	elevation (m)	natural fracture frequency	laboratory testing	notes and comments
	Rock coring started at 3.4m below grade	**136.4**					
4	**CLAYEY SILT**, some sand, trace gravel, hard, brown and grey, moist (GLACIAL TILL)	3.4 R1 135.0	TCR = **100%** SCR = **32%** RQD = **19%**	136	n/a		**3.4 / 136.4 - 3.5 / 136.3m:** Rubblized **3.5 / 136.3 - 4.2 / 135.5m:** Highly weathered, soil-like matrix
5	**QUEENSTON FORMATION** **Shale**, reddish brown, thinly bedded to medium bedded, weak, grey-green banding; joints are horizontal, closed to gapped, clean to sandy / silty filling	4.8 R2 133.5	TCR = **100%** SCR = **95%** RQD = **80%**	135	2+RZ 2 3 2+RZ 2 1 1	El. 134.5m: UCS = 33.3 MPa γ = 26.1 kN/m^3	**4.4 / 135.4 - 4.5 / 135.2m:** Rubblized **5.2 / 134.6 - 5.2 / 134.6m:** Rubblized
6				134			
7	**... at 6.7 m (Elev. 133.1 m), transition to sound rock**	6.3 R3 131.9	TCR = **100%** SCR = **93%** RQD = **83%**	133	4+SM 1 1 0 0		**6.4 / 133.3m:** Clay seam (35 mm) **6.6 / 133.2 - 6.6 / 133.2m:** CS **6.6 / 133.1 - 6.7 / 133.1m:** CS
8		7.9 R4 130.4	TCR = **100%** SCR = **95%** RQD = **88%**	132	4 1+SM 0		**8.2 / 131.6m:** Clay seam (25 mm)
9				131	0 0 0 0		
10		9.4 R5 128.9	TCR = **100%** SCR = **100%** RQD = **100%**	130	0 0 1 1		
11		10.9 R6 127.4	TCR = **100%** SCR = **100%** RQD = **95%**	129	0 0 0 1 1		
12				128			
13		12.4 R7 125.8	TCR = **100%** SCR = **100%** RQD = **95%**	127	0 0 1+SM 0		**13.2 / 126.6m:** Clay seam (10 mm)
		14.0m		126	0		

END OF COREHOLE

shale weathering zones: Z1 Z2 Z3 Z4

UCS (MPa) ● 5 25 50 100 250

estimated strength: R1 R2 R3 R4 R5 R6

GROUNDED ENGINEERING

Date Started : Apr 5, 2022
Position : E: 598068, N: 4805203 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 103

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

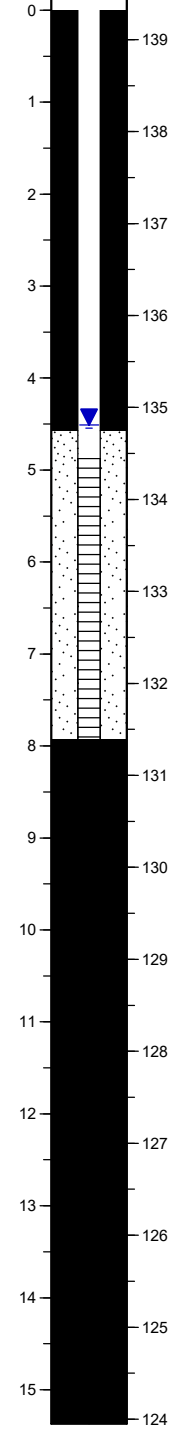
drill method : CME 75

elev depth (m)	description	graphic log	number	type	SPT N-value
139.3	**GROUND SURFACE**				
	140mm **ASPHALT**				
	FILL, sandy silt, some gravel, trace clay, trace asphalt, trace brick fragments, trace rock fragments, loose to compact, brown and dark brown, moist		1	SS	24
			2	SS	7
	...at 0.8 m, clayey silt, some sand, brown				
137.8					
1.5	**CLAYEY SILT**, some sand, trace gravel, trace rock fragments, very stiff, brown, moist (GLACIAL TILL)		3	SS	25
	...at 2.3 m, layered		4	SS	26
	...at 3.0 m, reddish brown, hard		5	SS	43
135.3			6	SS	60 / 25mm
4.0	...at 3.8 m, sandy, some gravel, some clay, trace rock fragments (shale fragments), very dense		1	RUN	
	QUEENSTON FORMATION (See rock core log for details)		2	RUN	
			3	RUN	
			4	RUN	
			5	RUN	
			6	RUN	
			7	RUN	
			8	RUN	
123.9					
15.4					

hollow stem augers (med) OD=175 mm

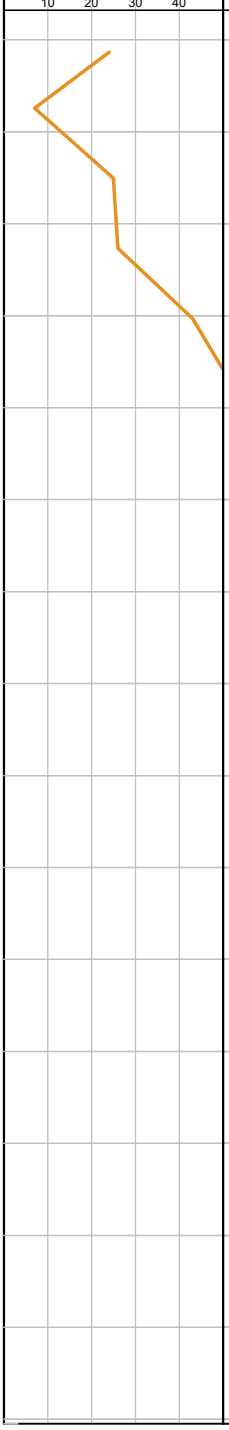
Rock coring (HQ) OD=96 mm

depth scale (m) / well details / elevation (m)



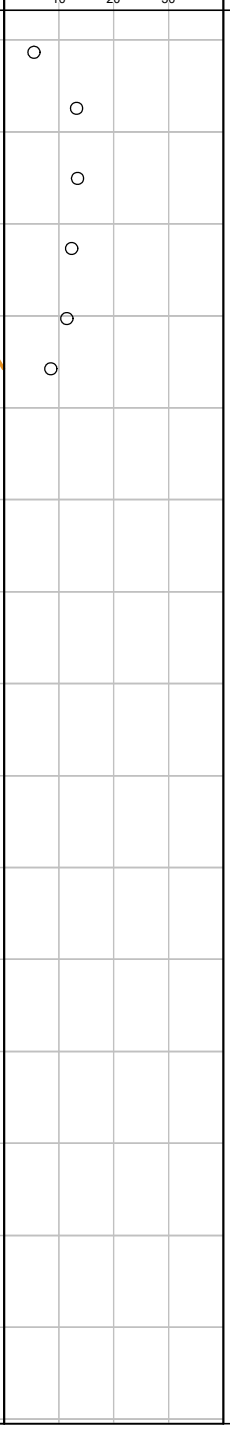
undrained shear strength (kPa)
○ unconfined + field vane
● pocket penetrometer ■ Lab Vane
40 80 120 160

SPT N-values (bpf)
× dynamic cone
10 20 30 40



headspace vapour (ppm)
× hexane □ isobutylene
⊠ methane
100 200 300

moisture / plasticity
PL MC LL
10 20 30



unstabilized water level

lab data and comments

grain size distribution (%) (MIT)
GR SA SI CL

...at 6.9 m, top of sound bedrock

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
May 26, 2022	4.5	134.8

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 5, 2022
Position : E: 598068, N: 4805203 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 103

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

Rock coring started at 4.0m below grade

QUEENSTON FORMATION
Shale, reddish brown, thinly bedded to medium bedded, weak, grey-green banding; joints are horizontal, closed to gapped, clean to sandy / silty filling

Run	elev / depth (m)	recovery
	135.3	
R1	4.0 / 134.6	TCR = **100%** SCR = **46%** RQD = **21%**
R2	4.7 / 133.1	TCR = **100%** SCR = **83%** RQD = **67%**
R3	6.2 / 131.6	TCR = **100%** SCR = **85%** RQD = **83%**
R4	7.7 / 130.0	TCR = **100%** SCR = **97%** RQD = **80%**
R5	9.3 / 128.5	TCR = **100%** SCR = **100%** RQD = **88%**
R6	10.8 / 127.0	TCR = **100%** SCR = **100%** RQD = **100%**
R7	12.3 / 125.5	TCR = **100%** SCR = **100%** RQD = **100%**
R8	13.8 / 123.9	TCR = **100%** SCR = **100%** RQD = **100%**
	15.4m	

... at 6.9 m (Elev. 132.4 m), transition to sound rock

END OF COREHOLE

elevation (m)	natural fracture frequency
135	3+RZ
	4
	1
	2+RZ
134	1
	3
	1+SM
	2
133	1+SM
	2
	0
132	2
	3+SM
	1
131	2+SM
	3
	0
130	1
	1
	2
129	1
	0
	0
	0
128	0
	1
	0
127	0
	0
	0
126	0
	1
	0
125	1
	0
	0
124	0

El. 132.1m:
UCS = 31.1 MPa
E = 5.30 GPa
γ = 26.1 kN/m^3

notes and comments

4.0 / 135.3 - 4.1 / 135.2m: Rubblized
4.3 / 135.0m: JT D UN R GA CT
5.3 / 134.0 - 5.3 / 134.0m: Rubblized
5.8 / 133.5m: JT D PR GA CT
6.1 / 133.2 - 6.1 / 133.2m: Weathered zone
6.1 / 133.2m: Clay seam (75 mm)
6.2 / 133.1 - 6.2 / 133.1m: Weathered zone
6.2 / 133.1 - 6.4 / 133.0m: Weathered zone
6.5 / 132.8m: Clay seam (50 mm)
6.8 / 132.5 - 6.9 / 132.5m: CS
7.8 / 131.5m: Clay seam (10 mm)
8.5 / 130.8m: Clay seam (10 mm)
8.5 / 130.8m: CS
8.7 / 130.6m: CS

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 7, 2022
Position : E: 598086, N: 4805289 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 104

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

drill method : CME 55

hollow stem augers (med) OD=175 mm

Rock coring (HQ) OD=96 mm

elev depth (m)	description	graphic log	number	type	SPT N-value
139.7	**GROUND SURFACE**				
	75mm **TOPSOIL**		1	SS	11
	FILL, clayey silt, some sand, trace gravel, trace rootlets, trace wood fragments, organic odour, stiff to very stiff, brown and dark brown, moist		2	SS	27
	...at 0.8 m, brown and reddish brown				
138.2 / 1.5	**CLAYEY SILT**, some sand, trace gravel, very stiff, brown and reddish brown, moist (GLACIAL TILL)		3	SS	27
	...at 2.3 m, some gravel, stiff, layered		4	SS	10
136.7 / 3.0	**SANDY SILT**, some gravel, some clay, trace rock fragments, (shale fragments), dense to very dense, reddish brown, moist (GLACIAL TILL)		5	SS	43
135.7 / 4.0	**QUEENSTON FORMATION** (See rock core log for details)		6	SS	50 / 0mm
			1	RUN	
			2	RUN	
			3	RUN	
			4	RUN	
			5	RUN	
			6	RUN	
			7	RUN	
125.6 / 14.1	**END OF BOREHOLE**				

depth scale (m) | well details | elevation (m)

undrained shear strength (kPa): unconfined, field vane, pocket penetrometer, Lab Vane; 40 80 120 160
SPT N-values (bpf): dynamic cone; 10 20 30 40
headspace vapour (ppm): hexane, isobutylene, methane; 100 200 300
moisture / plasticity: PL MC LL; 10 20 30
unstabilized water level

lab data and comments
grain size distribution (%) (MIT)
GR SA SI CL

13 15 45 27

LL=41.7

...at 7.8 m, top of sound bedrock

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
May 26, 2022	4.5	135.2

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 7, 2022
Position : E: 598086, N: 4805289 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 104

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

Rock coring started at 4.0m below grade (135.7)

QUEENSTON FORMATION
Shale, reddish brown, thinly bedded to medium bedded, weak, grey-green banding; joints are horizontal, closed to gapped, clean to sandy / silty filling

Run	elev. depth (m)	recovery
	4.0	
R1		TCR = **100%** SCR = **22%** RQD = **14%**
	134.8	
	4.9	
R2		TCR = **100%** SCR = **72%** RQD = **20%**
	133.3	
	6.4	
R3		TCR = **100%** SCR = **95%** RQD = **73%**
	131.8	
	7.9	
R4		TCR = **100%** SCR = **100%** RQD = **100%**
	130.3	
	9.4	
R5		TCR = **100%** SCR = **100%** RQD = **100%**
	128.7	
	11.0	
R6		TCR = **100%** SCR = **100%** RQD = **100%**
	127.2	
	12.5	
R7		TCR = **100%** SCR = **100%** RQD = **100%**
	125.6	

... at 7.8 m (Elev. 131.9 m), transition to sound rock

14.1m

END OF COREHOLE

elevation (m)	natural fracture frequency
	1+RZ
	4+CS
135	4+RZ
	5+IS
	3
	2
134	3+CS
	1
	2
133	1
	1
	2
132	3+IS
	0
	1
131	0
	0
	0
	1
130	1
	1
	0
129	0
	0
	0
	0
128	0
	1
127	0
	0
	1
126	0
	0

El. 127.4m:
UCS = 44.1 MPa
E = 10.10 GPa
γ = 26.2 kN/m^3

notes and comments

- **4.0 / 135.7 - 4.2 / 135.5m:** Clayey and Rubblized
- **4.3 / 135.4m:** Clay seam (25 mm)
- **4.5 / 135.2m:** Clay seam (10 mm)
- **4.5 / 135.1m:** Clay seam (15 mm)
- **4.7 / 135.0 - 4.7 / 134.9m:** Rubblized
- **5.0 / 134.7m:** IS clay; 10 mm
- **5.2 / 134.5 - 5.2 / 134.4m:** CS
- **5.8 / 133.9 - 5.9 / 133.8m:** Rubblized wth clay seam, 75 mm
- **5.9 / 133.7 - 6.0 / 133.7m:** Weathered zone
- **7.3 / 132.4 - 7.3 / 132.4m:** CS
- **7.7 / 131.9m:** IS clay; 25 mm

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 8, 2022
Position : E: 598115, N: 4805268 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 105

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

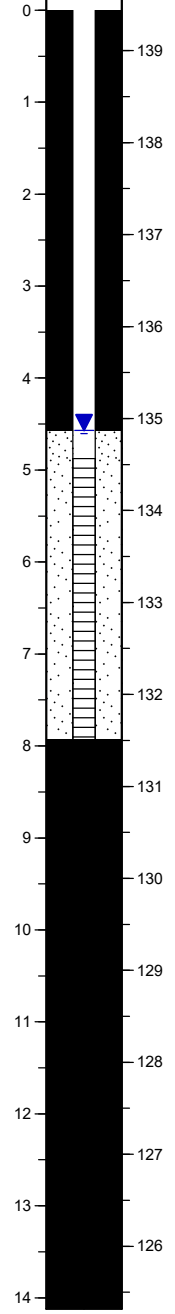
drill method : CME 55

elev depth (m)	description	graphic log	samples: number	type	SPT N-value
139.4	**GROUND SURFACE**				
	75mm **TOPSOIL**		1	SS	7
	FILL, clayey silt, some sand, trace gravel, trace rootlets, trace wood fragments, organic odour, firm, brown and dark brown, moist		2	SS	8
137.9 / 1.5	**CLAYEY SILT**, some sand, trace gravel, very stiff to hard, brown, moist (GLACIAL TILL)		3	SS	15
	...at 2.3 m, trace rock fragments, brown and grey		4	SS	39
136.4 / 3.0	**SANDY SILT**, some gravel, some clay, very dense, reddish brown, moist (GLACIAL TILL)		5A 5B	SS	94
135.7 / 3.7	...at 3.2 m, trace rock fragments (shale fragments)				
	QUEENSTON FORMATION (See rock core log for details)		1	RUN	
			2	RUN	
			3	RUN	
			4	RUN	
			5	RUN	
			6	RUN	
			7	RUN	
125.3 / 14.1	**END OF BOREHOLE**				

hollow stem augers (med) OD=175 mm

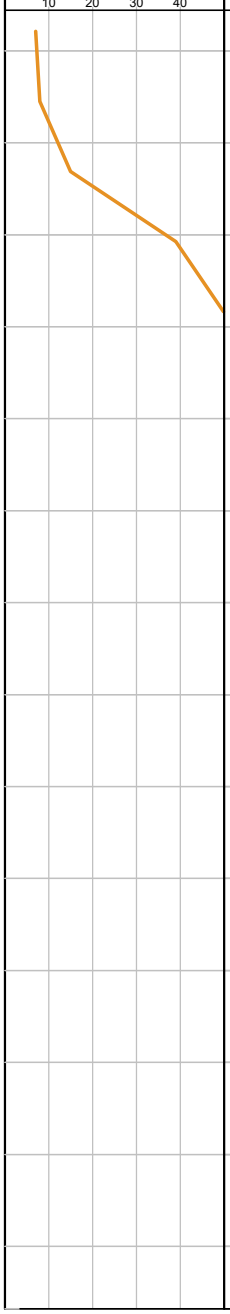
Rock coring (HQ) OD=96 mm

depth scale (m) / well details / elevation (m)



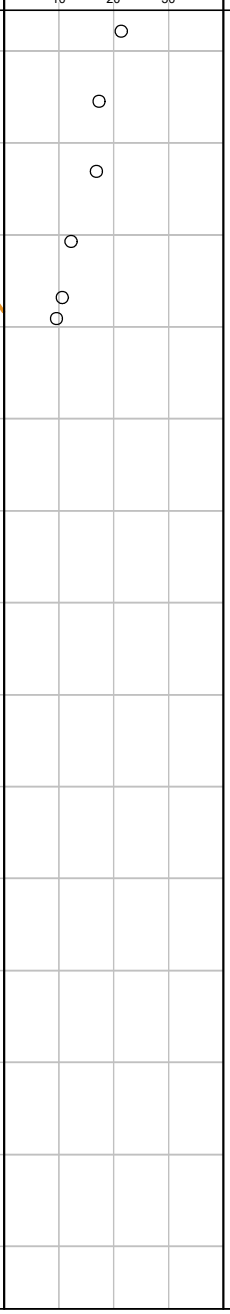
undrained shear strength (kPa)
○ unconfined + field vane
● pocket penetrometer ■ Lab Vane
40 80 120 160

SPT N-values (bpf)
× dynamic cone
10 20 30 40



headspace vapour (ppm)
× hexane □ isobutylene
⊠ methane
100 200 300

moisture / plasticity
PL MC LL
10 20 30



unstabilized water level

lab data and comments

grain size distribution (%) (MIT)
GR SA SI CL

24 29 35 12

...at 7.6 m, top of sound bedrock

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
May 26, 2022	4.6	134.8

Borehole was filled with drill water upon completion of drilling.

50 mm dia. monitoring well installed.
No. 10 screen

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

GROUNDED ENGINEERING

Date Started : Apr 8, 2022
Position : E: 598115, N: 4805268 (UTM 17T)
Elev. Datum : Geodetic

ROCK CORE LOG 105

File No. : 22-063 **Project** : 1989 Appleby Line, Burlington, Ontario **Client** : Latch Developments

depth (m)	stratigraphy	Run	elev. depth (m)	recovery	elevation (m)	natural fracture frequency	laboratory testing	notes and comments	elevation (m)
	Rock coring started at 3.7m below grade		**135.7**						
4	**QUEENSTON FORMATION** **Shale**, reddish brown, thinly bedded to medium bedded, weak, grey-green banding; joints are horizontal, closed to gapped, clean to sandy / silty filling		3.7			**2+RZ**		**3.7 / 135.7 - 3.8 / 135.6m:** Rubblized **3.9 / 135.5 - 4.0 / 135.5m:** Rubblized **4.0 / 135.4 - 4.1 / 135.4m:** Weathered zone **4.2 / 135.3 - 4.3 / 135.1m:** Weathered zone	
		R1		TCR = **100%** SCR = **60%** RQD = **42%**	135	**3+RZ**		**4.4 / 135.0m:** Clay seam (10 mm)	135
5			134.4			**3+SM**		**4.7 / 134.7m:** JT PR GA CT **4.9 / 134.5m:** IS clay	
			5.0			**3+IS**		**5.1 / 134.3 - 5.1 / 134.3m:** Rubblized	
		R2		TCR = **100%** SCR = **78%** RQD = **70%**	134	**2+RZ**			134
6			132.9			**1**		**5.7 / 133.8 - 5.7 / 133.7m:** Weathered zone **5.8 / 133.7 - 5.8 / 133.6m:** Rubblized	
			6.5			**4+SM**		**5.8 / 133.6 - 5.9 / 133.5m:** Clay seam (100 mm)	
		R3		TCR = **100%** SCR = **88%** RQD = **65%**	133	**2+SM**		**6.2 / 133.3m:** JT D UN GA CT	133
7			131.4			**1**			
			8.0			**4**			
		R4		TCR = **100%** SCR = **100%** RQD = **100%**	132	**2**		**7.5 / 132.0 - 7.6 / 131.8m:** Clay seam (125 mm)	132
8	**... at 7.6 m (Elev. 131.8 m), transition to sound rock**		129.9			**1**		**7.7 / 131.8m:** JT UN GA CT **7.8 / 131.6m:** JT UN GA CT	
			9.5			**4+SM**		**7.8 / 131.6m:** JT UN GA CT **7.9 / 131.6m:** JT UN GA CT	
		R5		TCR = **100%** SCR = **100%** RQD = **100%**	131	**3**			131
9			128.3			**1**			
			11.1			**0**			
		R6		TCR = **100%** SCR = **100%** RQD = **100%**	130	**0**			130
10			126.8			**1**			
			12.6			**1**			
		R7		TCR = **100%** SCR = **100%** RQD = **100%**	129	**0**			129
11			125.3			**1**			
			14.1m			**0**			
						0			
12					128	**0**	**El. 128.7m:** UCS = 22.5 MPa γ = 26.2 kN/m^3		128
						1			
						0			
13					127	**0**			127
						0			
						0			
14					126	**0**			126
						0			
	END OF COREHOLE								

shale weathering zones: Z1, Z2, Z3, Z4

UCS (MPa) ● 5 25 50 100 250

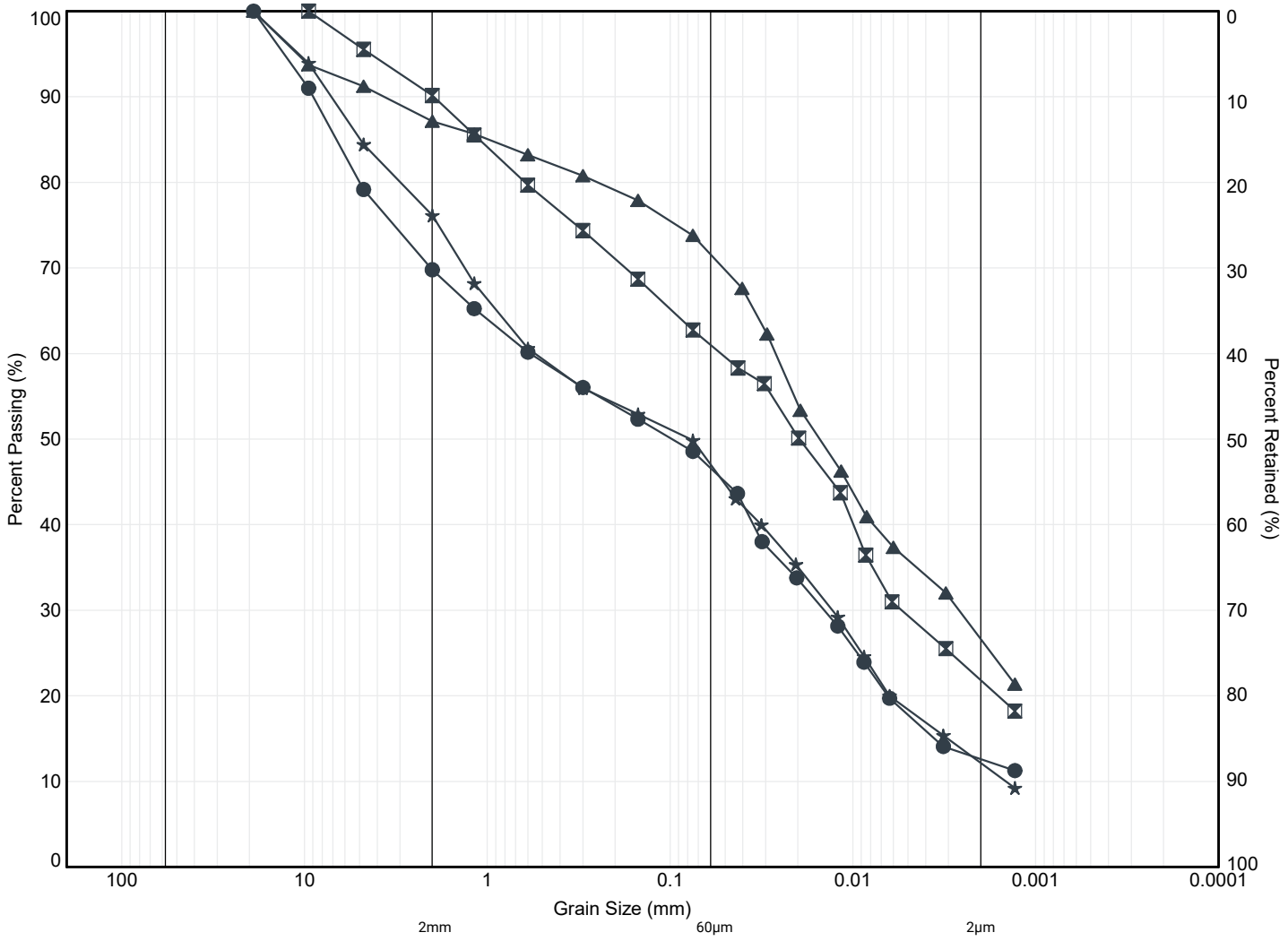
estimated strength: R1, R2, R3, R4, R5, R6

file: 22-063 gint.gpj

Tech : JAW | **PM** : KF | **Rev** : MD

APPENDIX B





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

MIT SYSTEM

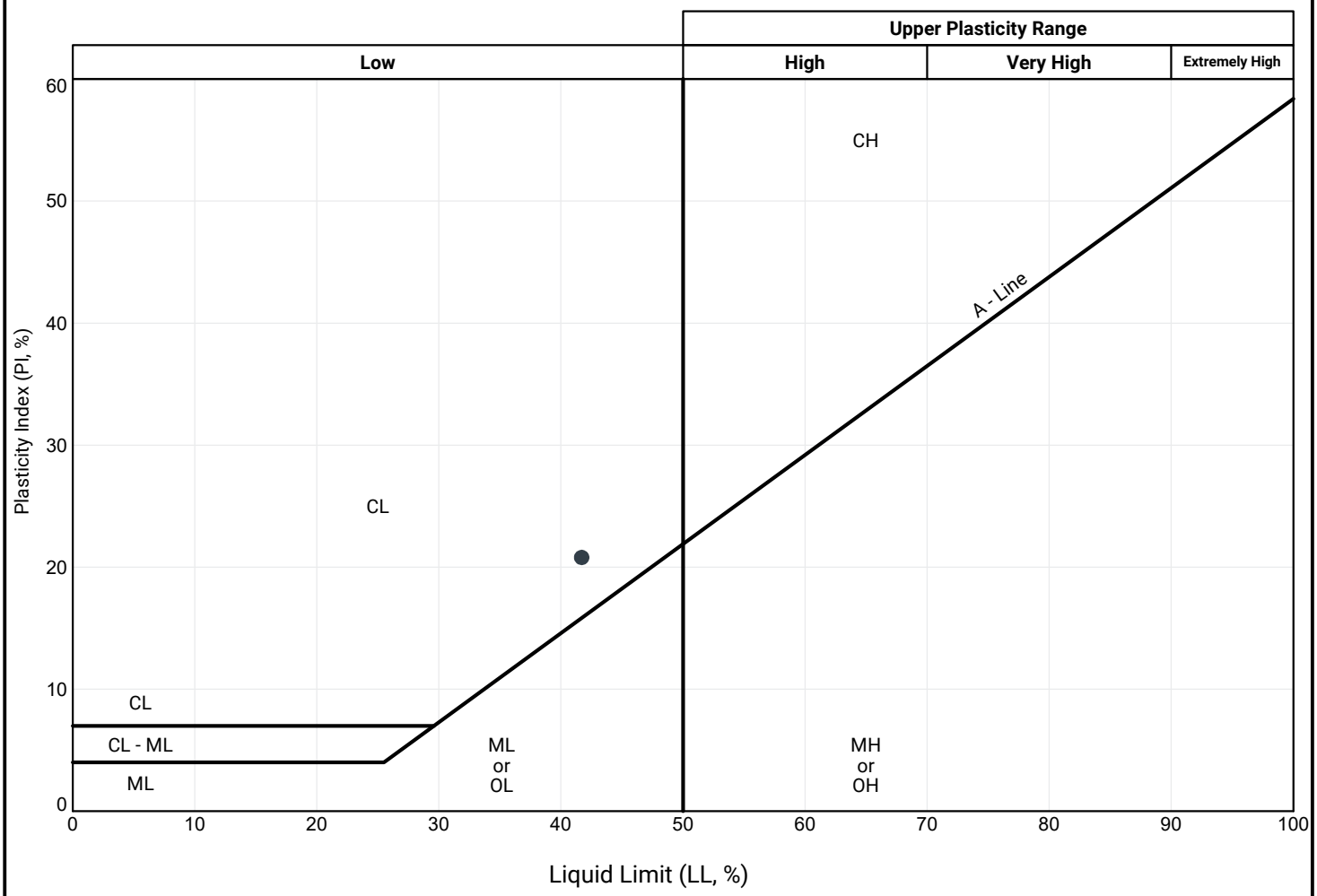
	Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**BH101**	SS3	3.1	136.5	30	23	34	13
⊠	**BH102**	SS4A	3.1	136.6	10	29	39	22
▲	**BH104**	SS4	2.5	137.2	13	15	45	27
★	**BH105**	SS5B	3.4	136.1	24	29	35	12

GROUNDED ENGINEERING

Title: **GRAIN SIZE DISTRIBUTION**

File No.: **22-063**

file: 22-063 gint.gpj



	Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)
●	**BH104**	SS4	2.5	137.2	42	21	21

GROUNDED ENGINEERING

Title: **ATTERBERG LIMITS CHART**

File No.: **22-063**

file: 22-063 gint.gpj

APPENDIX C



Grounded Engineering Inc
ATTN: Kim Falkenstein
1 Banigan Drive
TORONTO ON M4H 1G3

Date Received: 13-APR-22
Report Date: 21-APR-22 10:35 (MT)
Version: FINAL

Client Phone: 647-264-7943

Certificate of Analysis

Lab Work Order #: L2698995

Project P.O. #: NOT SUBMITTED
Job Reference: 22-063
C of C Numbers:
Legal Site Desc:

Amanda Overholster
Account Manager

[This report shall not be reproduced except in full without the written authority of the Laboratory.]

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2698995-1 BH102 SS1 Sampled By: CLIENT on 11-APR-22 @ 15:30 Matrix: SOIL							
Physical Tests							
Conductivity	0.171		0.0040	mS/cm		19-APR-22	R5765880
% Moisture	12.9		0.25	%	14-APR-22	14-APR-22	R5762584
pH	7.50		0.10	pH units		14-APR-22	R5763010
Redox Potential	297		-1000	mV		18-APR-22	R5764972
Resistivity	5830		1.0	ohm*cm		19-APR-22	
Leachable Anions & Nutrients							
Chloride	15.2		5.0	ug/g	19-APR-22	19-APR-22	R5766621
Anions and Nutrients							
Sulphate	49		20	ug/g	19-APR-22	19-APR-22	R5766621
Inorganic Parameters							
Acid Volatile Sulphides	0.73		0.20	mg/kg	18-APR-22	18-APR-22	R5764971
L2698995-2 BH103 SS6 Sampled By: CLIENT on 11-APR-22 @ 15:30 Matrix: SOIL							
Physical Tests							
Conductivity	0.261		0.0040	mS/cm		19-APR-22	R5765880
% Moisture	9.80		0.25	%	14-APR-22	14-APR-22	R5762584
pH	7.87		0.10	pH units		14-APR-22	R5763010
Redox Potential	292		-1000	mV		18-APR-22	R5764972
Resistivity	3830		1.0	ohm*cm		19-APR-22	
Leachable Anions & Nutrients							
Chloride	111		5.0	ug/g	19-APR-22	19-APR-22	R5766621
Anions and Nutrients							
Sulphate	40		20	ug/g	19-APR-22	19-APR-22	R5766621
Inorganic Parameters							
Acid Volatile Sulphides	1.46		0.20	mg/kg	18-APR-22	18-APR-22	R5764971
L2698995-3 BH104 SS4 Sampled By: CLIENT on 11-APR-22 @ 15:30 Matrix: SOIL							
Physical Tests							
Conductivity	0.197		0.0040	mS/cm		19-APR-22	R5765880
% Moisture	15.2		0.25	%	14-APR-22	14-APR-22	R5762584
pH	7.54		0.10	pH units		14-APR-22	R5763010
Redox Potential	295		-1000	mV		18-APR-22	R5764972
Resistivity	5080		1.0	ohm*cm		19-APR-22	
Leachable Anions & Nutrients							
Chloride	17.1		5.0	ug/g	19-APR-22	19-APR-22	R5766621
Anions and Nutrients							
Sulphate	66		20	ug/g	19-APR-22	19-APR-22	R5766621
Inorganic Parameters							
Acid Volatile Sulphides	0.75		0.20	mg/kg	18-APR-22	18-APR-22	R5764971

* Refer to Referenced Information for Qualifiers (if any) and Methodology.

Reference Information

Test Method References:

ALS Test Code	Matrix	Test Description	Method Reference**
CL-R511-WT	Soil	Chloride-O.Reg 153/04 (July 2011)	EPA 300.0

5 grams of dried soil is mixed with 10 grams of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011 and as of November 30, 2020), unless a subset of the Analytical Test Group (ATG) has been requested (the Protocol states that all analytes in an ATG must be reported).

EC-WT	Soil	Conductivity (EC)	MOEE E3138

A representative subsample is tumbled with de-ionized (DI) water. The ratio of water to soil is 2:1 v/w. After tumbling the sample is then analyzed by a conductivity meter.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

MOISTURE-WT	Soil	% Moisture	CCME PHC in Soil - Tier 1 (mod)
PH-WT	Soil	pH	MOEE E3137A

A minimum 10g portion of the sample is extracted with 20mL of 0.01M calcium chloride solution by shaking for at least 30 minutes. The aqueous layer is separated from the soil and then analyzed using a pH meter and electrode.

Analysis conducted in accordance with the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act (July 1, 2011).

REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580

This analysis is carried out in accordance with the procedure described in the "APHA" method 2580 "Oxidation-Reduction Potential" 2012. Samples are extracted at a fixed ratio with DI water. Results are reported as observed oxidation-reduction potential of the platinum metal-reference electrode employed, in mV.

RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B

"Soil Resistivity (calculated)" is determined as the inverse of the conductivity of a 2:1 water:soil leachate (dry weight). This method is intended as a rapid approximation for Soil Resistivity. Where high accuracy results are required, direct measurement of Soil Resistivity by the Wenner Four-Electrode Method (ASTM G57) is recommended.

SO4-WT	Soil	Sulphate	EPA 300.0

5 grams of soil is mixed with 50 mL of distilled water for a minimum of 30 minutes. The extract is filtered and analyzed by ion chromatography.

SULPHIDE-WT	Soil	Sulphide, Acid Volatile	APHA 4500S2J

This analysis is carried out in accordance with the method described in APHA 4500 S2-J. Hydrochloric acid is added to sediment samples within a purge and trap system. The evolved hydrogen sulphide (H2S) is carried into a basic solution by inert gas. The acid volatile sulfide is then determined colourimetrically.

** ALS test methods may incorporate modifications from specified reference methods to improve performance.

The last two letters of the above test code(s) indicate the laboratory that performed analytical analysis for that test. Refer to the list below:

Laboratory Definition Code	Laboratory Location
WT	ALS ENVIRONMENTAL - WATERLOO, ONTARIO, CANADA

Chain of Custody Numbers:

Reference Information

GLOSSARY OF REPORT TERMS

Surrogates are compounds that are similar in behaviour to target analyte(s), but that do not normally occur in environmental samples. For applicable tests, surrogates are added to samples prior to analysis as a check on recovery. In reports that display the D.L. column, laboratory objectives for surrogates are listed there.
mg/kg - milligrams per kilogram based on dry weight of sample
mg/kg wwt - milligrams per kilogram based on wet weight of sample
mg/kg lwt - milligrams per kilogram based on lipid weight of sample
mg/L - unit of concentration based on volume, parts per million.
< - Less than.
D.L. - The reporting limit.
N/A - Result not available. Refer to qualifier code and definition for explanation.

Test results reported relate only to the samples as received by the laboratory.
UNLESS OTHERWISE STATED, ALL SAMPLES WERE RECEIVED IN ACCEPTABLE CONDITION.
Analytical results in unsigned test reports with the DRAFT watermark are subject to change, pending final QC review.

Quality Control Report

Workorder: L2698995 Report Date: 21-APR-22

Client: Grounded Engineering Inc
1 Banigan Drive
TORONTO ON M4H 1G3
Contact: Kim Falkenstein

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-R511-WT	**Soil**							
Batch R5766621								
WG3718162-3 CRM		**AN-CRM-WT**						
Chloride			99.5		%		70-130	19-APR-22
WG3718162-4 DUP		**WG3718162-5**						
Chloride		<5.0	<5.0	RPD-NA	ug/g	N/A	30	19-APR-22
WG3718162-2 LCS								
Chloride			101.9		%		80-120	19-APR-22
WG3718162-1 MB								
Chloride			<5.0		ug/g		5	19-APR-22
EC-WT	**Soil**							
Batch R5765880								
WG3718107-4 DUP		**WG3718107-3**						
Conductivity		0.192	0.190		mS/cm	1.1	20	19-APR-22
WG3718107-2 IRM		**WT SAR4**						
Conductivity			94.8		%		70-130	19-APR-22
WG3718314-1 LCS								
Conductivity			99.8		%		90-110	19-APR-22
WG3718107-1 MB								
Conductivity			<0.0040		mS/cm		0.004	19-APR-22
MOISTURE-WT	**Soil**							
Batch R5762584								
WG3717098-4 DUP		**L2698964-1**						
% Moisture		14.0	14.3		%	1.8	20	14-APR-22
WG3717098-2 LCS								
% Moisture			99.7		%		90-110	14-APR-22
WG3717098-1 MB								
% Moisture			<0.25		%		0.25	14-APR-22
PH-WT	**Soil**							
Batch R5763010								
WG3717135-1 DUP		**L2698973-12**						
pH		7.79	7.79	J	pH units	0.00	0.3	14-APR-22
WG3717231-1 LCS								
pH			7.04		pH units		6.9-7.1	14-APR-22
REDOX-POTENTIAL-WT	**Soil**							
Batch R5764972								
WG3717864-1 CRM		**WT-REDOX**						
Redox Potential			104.6		%		90-110	18-APR-22
WG3717594-1 DUP		**L2698995-1**						

Quality Control Report

Client: Grounded Engineering Inc
1 Banigan Drive
TORONTO ON M4H 1G3
Contact: Kim Falkenstein

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
REDOX-POTENTIAL-WT	**Soil**							
Batch R5764972								
WG3717594-1 DUP		**L2698995-1**						
Redox Potential		297	280		mV	5.9	25	18-APR-22
SO4-WT	**Soil**							
Batch R5766621								
WG3718162-3 CRM		**AN-CRM-WT**						
Sulphate			100.8		%		60-140	19-APR-22
WG3718162-4 DUP		**WG3718162-5**						
Sulphate		71	71		ug/g	0.6	25	19-APR-22
WG3718162-2 LCS								
Sulphate			102.4		%		70-130	19-APR-22
WG3718162-1 MB								
Sulphate			<20		ug/g		20	19-APR-22
SULPHIDE-WT	**Soil**							
Batch R5764971								
WG3717857-3 DUP		**L2698995-3**						
Acid Volatile Sulphides		0.75	1.15		mg/kg	42	45	18-APR-22
WG3717857-2 LCS								
Acid Volatile Sulphides			73.6		%		70-130	18-APR-22
WG3717857-1 MB								
Acid Volatile Sulphides			<0.20		mg/kg		0.2	18-APR-22

Quality Control Report

Workorder: L2698995 Report Date: 21-APR-22

Client: Grounded Engineering Inc
1 Banigan Drive
TORONTO ON M4H 1G3
Contact: Kim Falkenstein

Legend:

Limit	ALS Control Limit (Data Quality Objectives)
DUP	Duplicate
RPD	Relative Percent Difference
N/A	Not Available
LCS	Laboratory Control Sample
SRM	Standard Reference Material
MS	Matrix Spike
MSD	Matrix Spike Duplicate
ADE	Average Desorption Efficiency
MB	Method Blank
IRM	Internal Reference Material
CRM	Certified Reference Material
CCV	Continuing Calibration Verification
CVS	Calibration Verification Standard
LCSD	Laboratory Control Sample Duplicate

Sample Parameter Qualifier Definitions:

Qualifier	Description
J	Duplicate results and limits are expressed in terms of absolute difference.
RPD-NA	Relative Percent Difference Not Available due to result(s) being less than detection limit.

Hold Time Exceedances:

All test results reported with this submission were conducted within ALS recommended hold times.

ALS recommended hold times may vary by province. They are assigned to meet known provincial and/or federal government requirements. In the absence of regulatory hold times, ALS establishes recommendations based on guidelines published by the US EPA, APHA Standard Methods, or Environment Canada (where available). For more information, please contact ALS.

The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide confidence in the accuracy of associated test results.

Please note that this report may contain QC results from anonymous Sample Duplicates and Matrix Spikes that do not originate from this Work Order.

Chain of Custody (COC) / Analytical Request Form

ALS www.alsglobal.com

Canada Toll Free: 1 800 668 9878

L2698995-COFC

COC Number: 17 -

Page of

Report To	Contact and company name below will appear on the final report
Company:	Grounded Engineering
Contact:	Kim Falkenstein
Phone:	647-264-7943
	Company address below will appear on the final report
Street:	1 Banigan Drive
City/Province:	Toronto, ON
Postal Code:	M4H 1G3
Invoice To	Same as Report To ☑ YES ☐ NO
	Copy of Invoice with Report ☑ YES ☐ NO
Company:	
Contact:	

Project Information	
ALS Account # / Quote #:	
Job #:	22-063
PO / AFE:	
LSD:	
ALS Lab Work Order # (lab use only):	L2698995
ALS Sample # (lab use only)	

Report Format / Distribution	
Select Report Format:	☑ PDF ☑ EXCEL ☐ EDD (DIGITAL)
Quality Control (QC) Report with Report	☑ YES ☐ NO
☐ Compare Results to Criteria on Report - provide details below if box checked	
Select Distribution:	☑ EMAIL ☐ MAIL ☐ FAX
Email 1 or Fax	kfalkenstein@groundedeng.ca
Email 2	
Email 3	

Invoice Distribution	
Select Invoice Distribution:	☐ EMAIL ☐ MAIL ☐ FAX
Email 1 or Fax	
Email 2	

Oil and Gas Required Fields (client use)	
AFE/Cost Center:	PO#
Major/Minor Code:	Routing Code:
Requisitioner:	
Location:	
ALS Contact:	Sampler:

Select Service Level Below - Contact your AM to confirm all E&P TATs (surcharges may apply)

Standard TAT is 15 business days. DTOX analysis standard TAT is 5 business days

PRIORITY (Business Days)		EMERGENCY	
	15 day [R- Regular] ☑		5 Business day - DTOX [R - Regular] ☐
	10 day [P-50%] ☐		3 Business day - DTOX [E - 100%] ☐
	5 day [E-100%] ☐		

Date and Time Required for all E&P TATs: dd-mmm-yy hh:mm

For tests that can not be performed according to the service level selected, you will be contacted.

Analysis Request

Indicate Filtered (F), Preserved (P) or Filtered and Preserved (F/P) below

Sample Identification and/or Coordinates (This description will appear on the report)	Date (dd-mmm-yy)	Time (hh:mm)	Sample Type	NUMBER OF CONTAINERS	Corrosivity	TCLP	SAMPLES ON HOLD
~~BH1-SS2~~ BH102 SS1	11-04-2022	15:30:00 PM	Soil	1	X		
~~BH1-SS3~~ BH103 SS6	11-04-2022	15:30:00 PM	Soil	1	X		
~~BH1-SS6~~ BH104 SS4	11-04-2022	15:30:00 PM	Soil	1	X		
TCLP	11-04-2022	15:30:00 PM	Soil	1		X	

Drinking Water (DW) Samples[1] (client use)	Special Instructions / Specify Criteria to add on report by clicking on the drop-down list below (electronic COC only)
Are samples taken from a Regulated DW System? ☐ YES ☐ NO	
Are samples for human consumption/ use? ☐ YES ☐ NO	

SAMPLE CONDITION AS RECEIVED (lab use only)

Frozen ☐	SIF Observations	Yes ☐	No ☐
Ice Packs ☐ Ice Cubes ☐	Custody seal intact	Yes ☐	No ☐
Cooling Initiated ☐			

INITIAL COOLER TEMPERATURES °C				FINAL COOLER TEMPERATURES °C			
				12.5			

SHIPMENT RELEASE (client use)			INITIAL SHIPMENT RECEPTION (lab use only)		FINAL SHIPMENT RECEPTION (lab use only)		
Released by: Kim Falkenstein	Date: April 11h, 2022.	Time:	Received by:	Date:	Time:	Received by: FH	Date: 2022-04-13 Time: 11AM

REFER TO BACK PAGE FOR ALS LOCATIONS AND SAMPLING INFORMATION

WHITE - LABORATORY COPY YELLOW - CLIENT COPY

Failure to complete all portions of this form may delay analysis. Please fill in this form LEGIBLY. By the use of this form the user acknowledges and agrees with the Terms and Conditions as specified on the back page of the white - report copy.

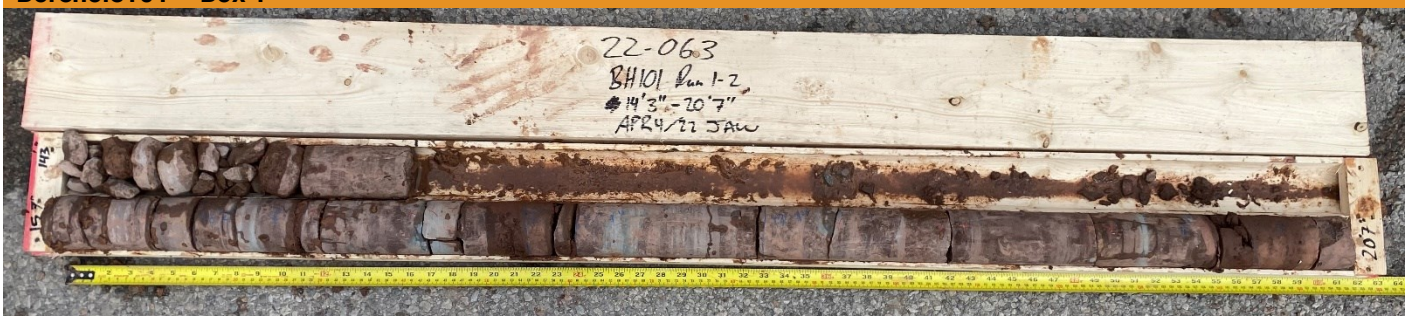
If any water samples are taken from a **Regulated Drinking Water (DW) System**, please submit using an **Authorized DW COC form.**

NOV 2015 FR

APPENDIX D



Borehole101 – Box 1



Depth: 4.3 to 6.3 m below grade (Elev. 135.2 to 133.2 m)

Borehole101 – Box 2



Depth: 6.3 to 9.3 m below grade (Elev. 133.2 to 130.2 m)

Borehole101 – Box 3



Depth: 9.3 to 12.3 m below grade (Elev. 130.2 to 127.2 m)

Borehole101 – Box 4



Depth: 12.3 to 15.4 m below grade (Elev. 127.2 to 124.1 m)

Borehole102 – Box 1



Depth: 3.8 to 6.4 m below grade (Elev. 136.0 to 133.4 m)

Borehole102 – Box 2



Depth: 6.4 to 9.4 m below grade (Elev. 133.4 to 130.4 m)

Borehole102 – Box 3



Depth: 9.4 to 12.4 m below grade (Elev. 130.4 to 127.4 m)

Borehole102 – Box 4



Depth: 12.4 to 14.0 m below grade (Elev. 127.4 to 125.8 m)

Borehole103 – Box 1



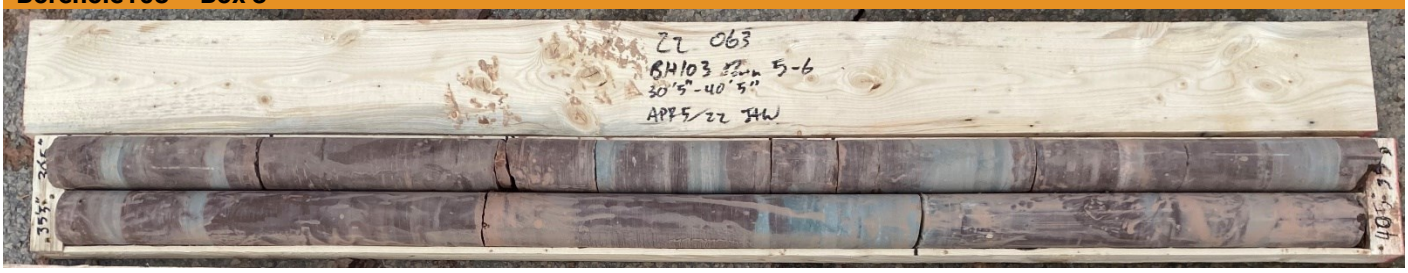
Depth: 3.8 to 6.2 m below grade (Elev. 135.5 to 133.1 m)

Borehole103 – Box 2



Depth: 6.2 to 9.3 m below grade (Elev. 133.1 to 130.0 m)

Borehole103 – Box 3



Depth: 9.3 to 12.3 m below grade (Elev. 130.0 to 127.0 m)

Borehole103 – Box 4



Depth: 12.3 to 15.4 m below grade (Elev. 127.0 to 123.9 m)

Borehole104 – Box 1



Depth: 4.0 to 6.4 m below grade (Elev. 135.7 to 133.3 m)

Borehole104 – Box 2



Depth: 6.4 to 9.4 m below grade (Elev. 133.3 to 130.3 m)

Borehole104 – Box 3



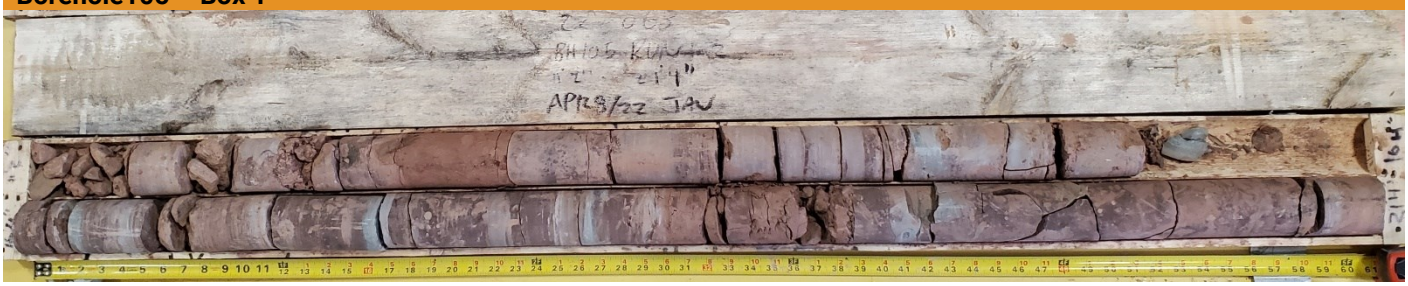
Depth: 9.4 to 12.5 m below grade (Elev. 130.3 to 127.2 m)

Borehole104 – Box 4



Depth: 12.5 to 14.1 m below grade (Elev. 127.2 to 125.6 m)

Borehole105 – Box 1



Depth: 3.4 to 6.5 m below grade (Elev. 136.0 to 132.9 m)

Borehole105 – Box 2



Depth: 6.5 to 9.5 m below grade (Elev. 132.9 to 129.9 m)

Borehole105 – Box 3



Depth: 9.5 to 12.6 m below grade (Elev. 129.9 to 126.8 m)

Borehole105 – Box 4



Depth: 12.6 to 14.1 m below grade (Elev. 126.8 to 125.3 m)

APPENDIX E



Geomechanica Inc.
Suite 900 – 390 Bay St.
Toronto Ontario
Canada M5H 2Y2

April 22, 2022

Ms. Kim Falkenstein
Grounded Engineering
1 Banigan Drive
Toronto, ON
Canada, M4H 1E9

Re: UCS Testing
(Grounded Project No. 21-063)

Dear Ms. Falkenstein:

On April 5th, 2022, five (5) HQ-sized core samples were received by Geomechanica Inc. via drop-off by Grounded personnel. These samples were identified as being from Grounded project 21-063. From these samples, five (5) Uniaxial Compressive Strength (UCS) test specimens were prepared and tested.

Details regarding the steps of specimen preparation and testing along with the test results are presented in the accompanying laboratory report and summary spreadsheet.

Sincerely,



Bryan Tatone Ph.D., P. Eng.

Geomechanica Inc.
Tel: (647) 478-9767
Email: bryan.tatone@geomechanica.com

GEOMECHANICA

Rock Laboratory Testing Results

A report submitted to:

Kim Falkenstein
Grounded Engineering Inc.
1 Banigan Drive
Toronto, Ontario
Canada, M4H 1G3

Prepared by:

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Omid Mahabadi, PhD, PEng
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Toronto ON
M5H 2Y2 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

April 22, 2022
Project number: 22-063

Abstract

This document summarizes the results of rock laboratory testing, including 5 Uniaxial Compressive Strength (UCS) tests. The UCS values and Young's modulus (for select specimens) along with photographs of samples before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests 1
Appendices 4

Disclaimer: This report was prepared by Geomechanica Inc. for Grounded Engineering Inc.. The material herein reflects Geomechanica Inc.'s best judgment given the information available at the time of preparation. Any use which a third party makes of this report, any reliance on or decision to be made based on it, are the responsibility of such third parties. Geomechanica Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica's rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial displacement rate of approximately 0.150 mm/min (Figure 1). The specimen preparation and testing procedure included the following:

1. Unwrapping the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting the core sample to obtain a cylindrical specimen with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding of the specimen to obtain flat (within ± 0.025 mm) and parallel end faces (within 0.25°).
4. Placing the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force and axial deformation to determine the peak strength (UCS) and tangent Young's modulus (for select specimens).



Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness

criteria, as checked with a feeler gauge, and the minimum length:diameter criteria were met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 with the following note:

- Testing of some specimens included measurement of the UCS and elastic modulus, but not the Poisson's ratio. This represents a hybrid between Methods C and D of ASTM D7012-14.

1.2 Results

The results of UCS testing are summarized in Table 1. The corresponding stress-strain curve is presented in Figure 2. The Young's modulus is the tangent modulus calculated as the slope of the best-fit line through 600 data points defining the stress-strain curve. Typically the modulus is defined at 50% of the UCS strength. However, due to non-linear pre-peak stress-strain behaviour in some specimens, custom stress ranges (where the specimens deformed linearly) were selected for moduli determination. These stress ranges along with additional specimen details and measurements are provided in the summary spreadsheet that accompanies this report.

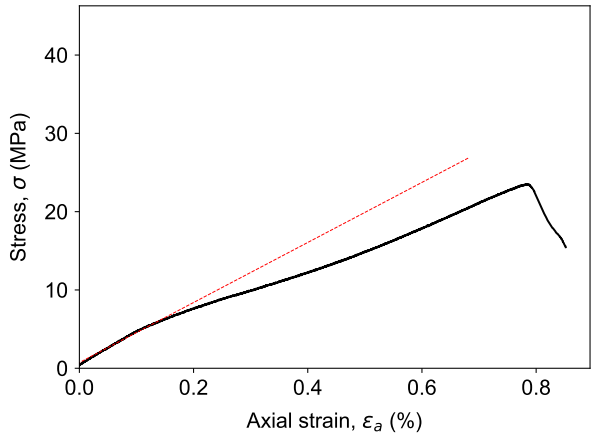
Table 1: Summary of UCS test results.

Sample	Depth (ft' in")	Bulk density ρ (g/cm^3)	UCS (MPa)	Young's modulus E (GPa)	Lithology	Failure description
BH101 Run 3, RC1	20' 9" - 21' 5"	2.643	23.5	3.8	Red shale	1
BH102 Run 2, RC2	17' 4" - 18' 5"	2.660	33.3	-	Red shale	1
BH103 Run 3, RC3	23' 7" - 24' 7"	2.656	31.1	5.3	Red shale	1, 2
BH104 Run 8, RC4	40' 2.5" - 41' 2"	2.672	44.1	10.1	Red shale with light-coloured reduction zones	3
BH105 Run 6, RC5	35' 4" - 36' 4"	2.667	22.5	-	Red shale with light-coloured reduction zones	3, 2

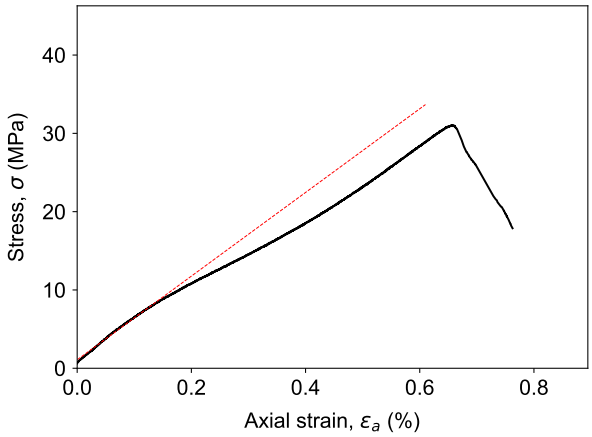
[1] Inclined shear fracture and axial splitting failure
[2] Specimen emitted pore water upon loading
[3] Axial splitting failure

1.3 Specimen photographs

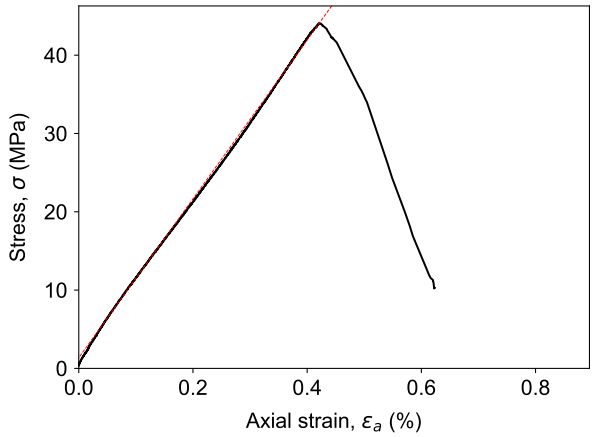
Photographs of the specimens before and after testing are presented in the Appendix of this report.



(a) BH101-Run-3-RC1



(b) BH103-Run-3-RC3



(c) BH104-Run-8-RC4

Figure 2: Measured stress-strain curves.

Appendices

Specimen sheets

- BH101 Run 3, RC1
- BH102 Run 2, RC2
- BH103 Run 3, RC3
- BH104 Run 8, RC4
- BH105 Run 6, RC5

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Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	22-063
Sample	BH101 Run 3, RC1	**Depth**	20’ 9” - 21’ 5”

Specimen parameters	
Diameter (mm) [a]	62.98
Length (mm) [a]	130.15
Bulk density ρ (g/cm^3)	2.643
UCS (MPa)	23.5
Young’s modulus E (GPa) [b]	3.8
Lithology	Red shale
Failure description [c]	1

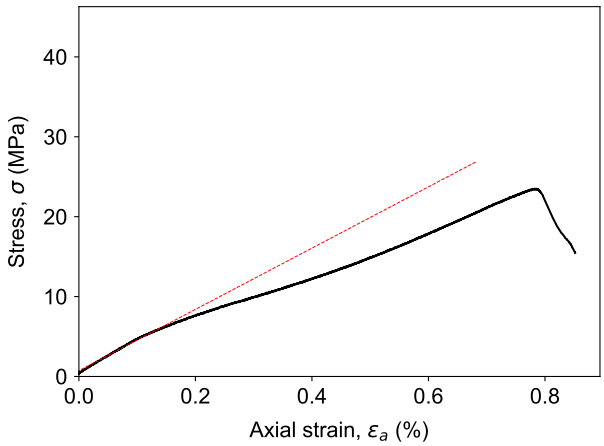
Prior to testing



After testing



[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 18.0% of the peak strength.
[c] Failure description: [1] Inclined shear fracture and axial splitting failure;



Remarks: Loading rate: 0.15 mm/min.

Performed by	HS/HS	**Date**	2022-04-21

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	22-063
Sample	BH102 Run 2, RC2	**Depth**	17’ 4” - 18’ 5”

Specimen parameters	
Diameter (mm) [a]	62.98
Length (mm) [a]	130.45
Bulk density ρ (g/cm^3)	2.660
UCS (MPa)	33.3
Lithology	Red shale
Failure description [b]	1

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [1] Inclined shear fracture and axial splitting failure;

Prior to testing



After testing



Remarks: Loading rate: 0.15 mm/min.

Performed by	HS/HS	**Date**	2022-04-21

GEOMECHANICA

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	22-063
Sample	BH103 Run 3, RC3	**Depth**	23’ 7” - 24’ 7”

Specimen parameters	
Diameter (mm) [a]	63.01
Length (mm) [a]	129.04
Bulk density ρ (g/cm^3)	2.656
UCS (MPa)	31.1
Young’s modulus E (GPa) [b]	5.3
Lithology	Red shale
Failure description [c]	1, 2

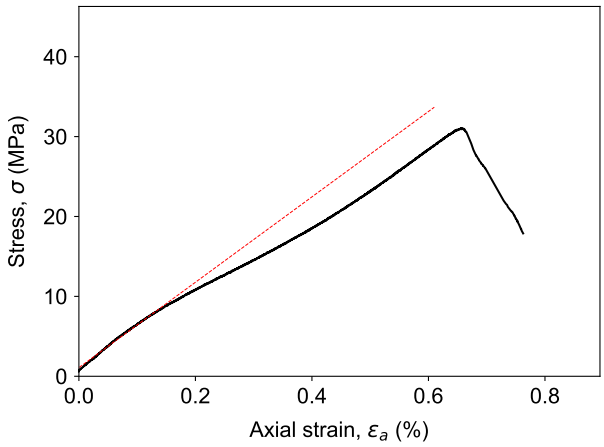
Prior to testing



After testing



[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through ±300 data points on either side of the point representing 18.0% of the peak strength.
[c] Failure description: [1] Inclined shear fracture and axial splitting failure; [2] Specimen emitted pore water upon loading;



Remarks: Loading rate: 0.15 mm/min.

Performed by	HS/HS	**Date**	2022-04-21

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	22-063
Sample	BH104 Run 8, RC4	**Depth**	40’ 2.5” - 41’ 2”

Specimen parameters

Diameter (mm) [a]	62.98
Length (mm) [a]	130.75
Bulk density ρ (g/cm^3)	2.672
UCS (MPa)	44.1
Young’s modulus E (GPa) [b]	10.1
Lithology	Red shale with light-coloured reduction zones
Failure description [c]	3

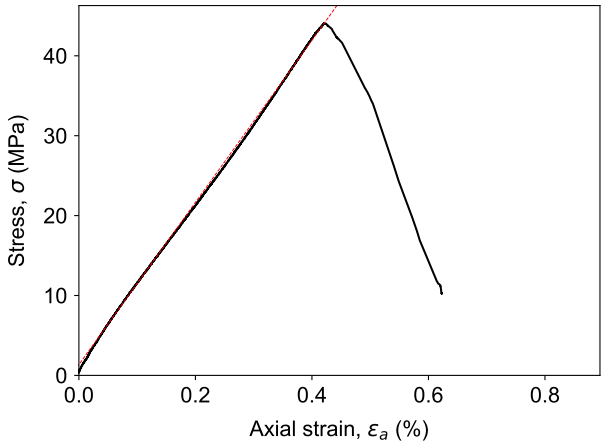
[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Tangent modulus, calculated as the slope of the best fit line through $\pm$300 data points on either side of the point representing 25.0% of the peak strength.
[c] Failure description: [3] Axial splitting failure;

Prior to testing



After testing





Remarks: Loading rate: 0.15 mm/min.

Performed by	HS/HS	**Date**	2022-04-21

GEOMECHANICA

Uniaxial Compression Test

Client	Grounded Engineering Inc.	**Project**	22-063
Sample	BH105 Run 6, RC5	**Depth**	35’ 4” - 36’ 4”

Specimen parameters	
Diameter (mm) [a]	62.99
Length (mm) [a]	130.37
Bulk density ρ (g/cm^3)	2.667
UCS (MPa)	22.5
Lithology	Red shale with light-coloured reduction zones
Failure description [b]	3, 2

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [3] Axial splitting failure; [2] Specimen emitted pore water upon loading;

Prior to testing



After testing

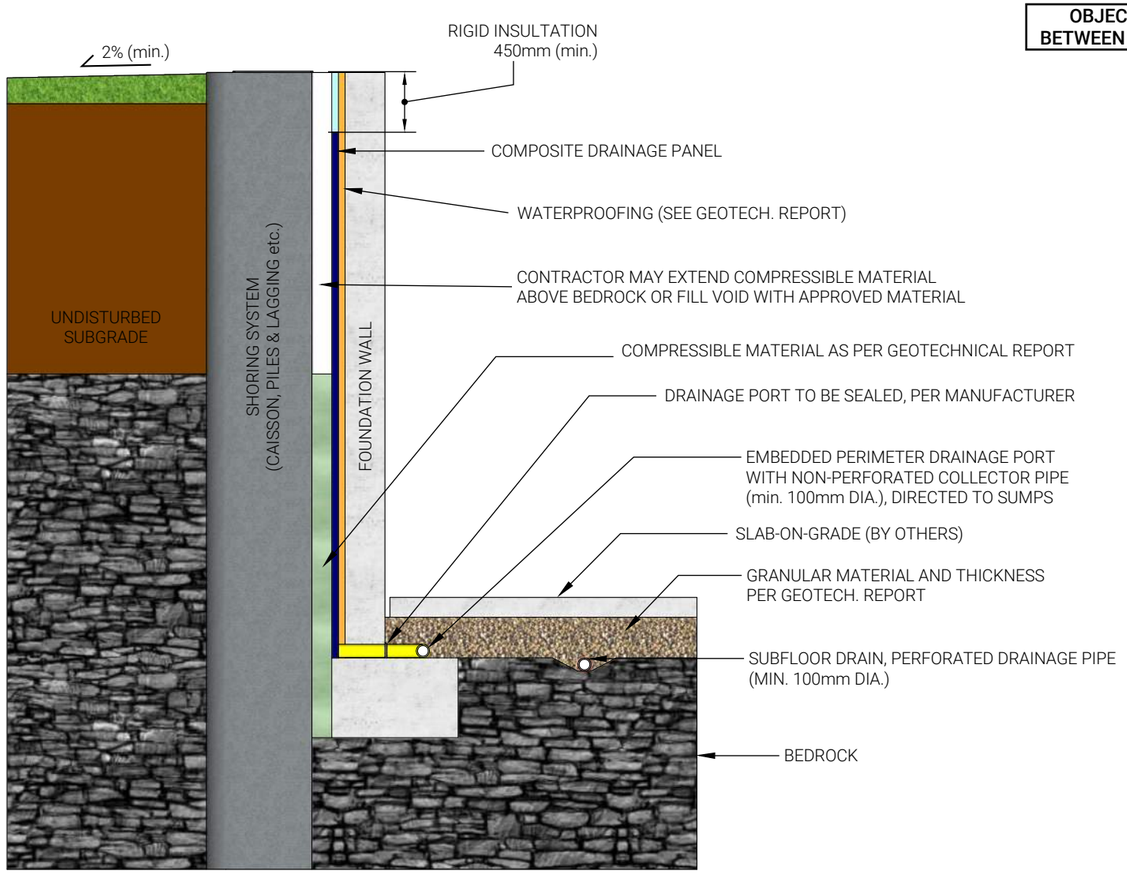


Remarks: Loading rate: 0.15 mm/min.

Performed by	HS/HS	**Date**	2022-04-21

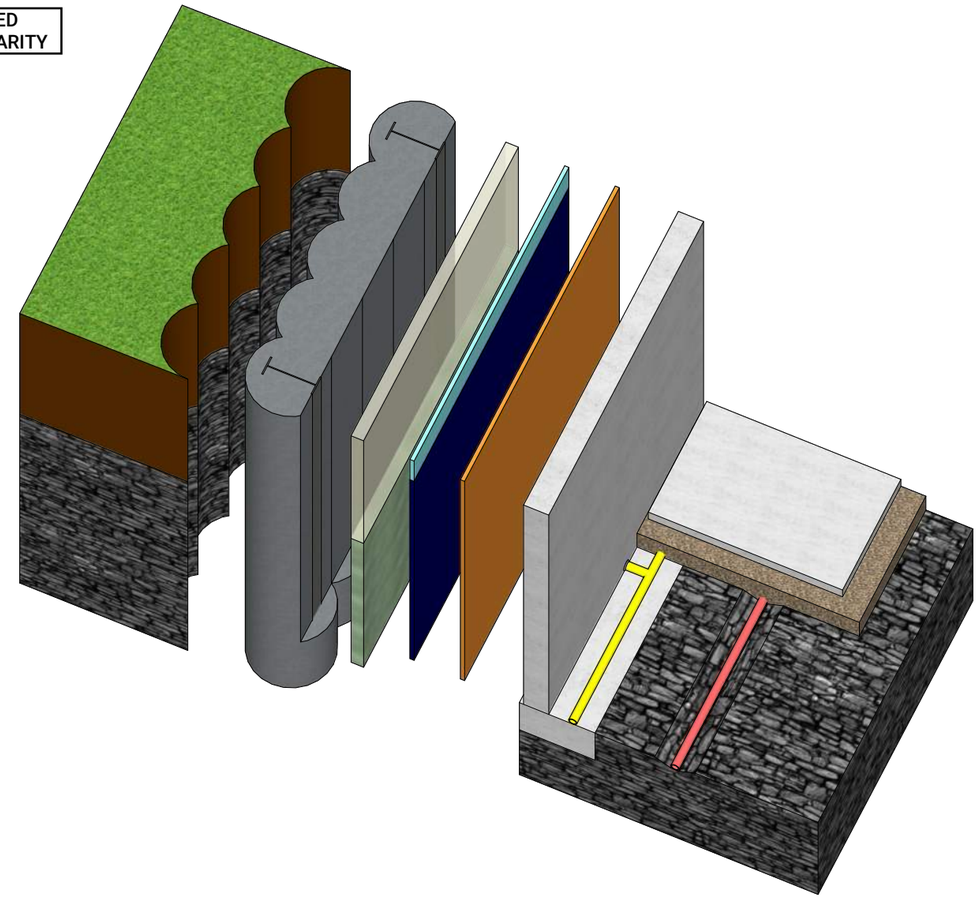
APPENDIX F





SECTIONAL VIEW

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



ISOMETERIC VIEW

SUBFLOOR DRAINAGE SYSTEM

1. THE SUBFLOOR DRAINS SHOULD BE SET IN PARALLEL ROWS, IN ONE DIRECTION, AND SPACED AS PER THE GEOTECHNICAL REPORT.
2. THE INVERT OF THE PIPES SHOULD BE A MINIMUM OF 300mm BELOW THE UNDERSIDE OF THE SLAB-ON-GRADE.
3. A CAPILLARY MOISTURE BARRIER (I.E. DRAINAGE LAYER) CONSISTING OF A MINIMUM 200 mm LAYER OF CLEAR STONE (OPSS MUNI 1004) COMPACTED TO A DENSE STATE (OR AS PER THE GEOTECHNICAL REPORT). WHERE VEHICULAR TRAFFIC IS REQUIRED, THE UPPER 50 mm OF THE CAPILLARY MOISTURE BARRIER MAY BE REPLACED WITH GRANULAR A (OPSS MUNI 1010) COMPACTED TO A MINIMUM 98% SPMDD.

PERIMETER DRAINAGE SYSTEM

1. FOR A DISTANCE OF 1.2m FROM THE BUILDING, THE GROUND SURFACE SHOULD HAVE A MINIMUM 2% GRADE.
2. PREFABRICATED COMPOSITE DRAINAGE PANEL (CONTINUOUS COVER, AS PER MANUFACTURER'S REQUIREMENTS) IS RECOMMENDED BETWEEN THE BASEMENT WALL AND RIGID SHORING WALL. THE DRAINAGE PANEL MAY CONSIST OF MIRADRAIN 6000 OR AN APPROVED EQUIVALENT.
3. PERIMETER DRAINAGE IS TO BE COLLECTED IN NON-PERFORATED PIPES AND CONVEYED DIRECTLY TO THE BUILDING SUMPS.
4. PERIMETER DRAINAGE PORTS SHOULD BE SPACED A MAXIMUM 3m ON-CENTRE. EACH PORT SHOULD HAVE A MINIMUM CROSS-SECTIONAL AREA OF 1500 mm2.

GENERAL NOTES

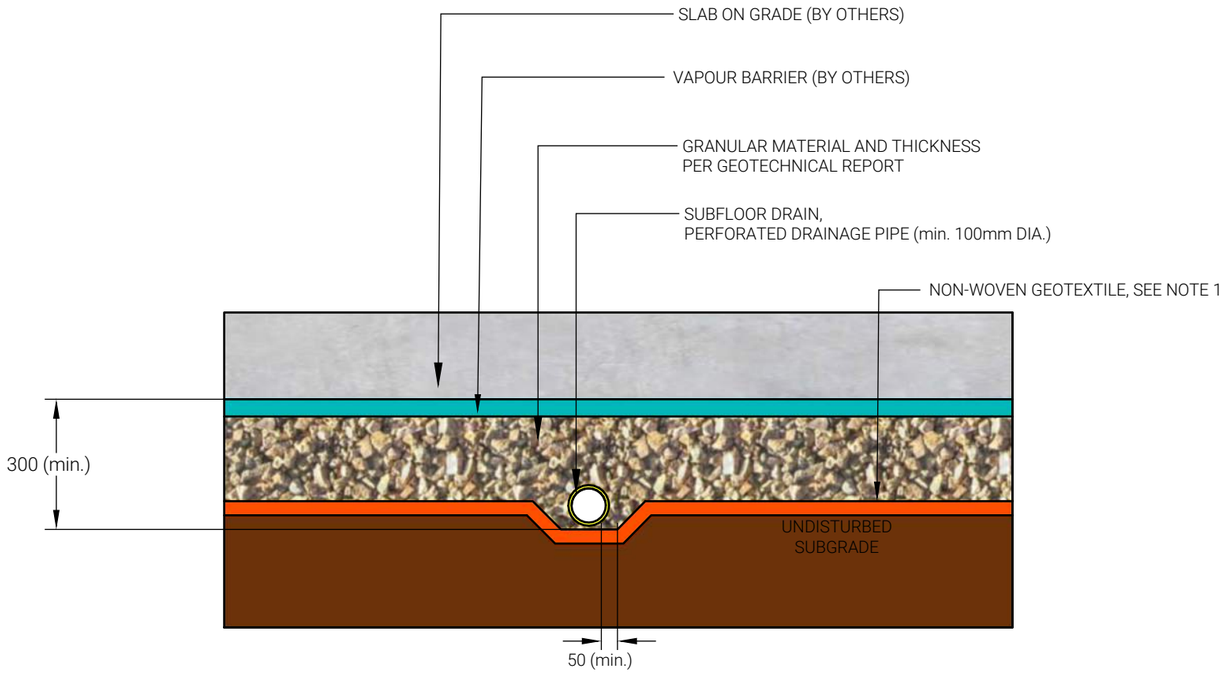
1. THERE SHOULD BE NO STRUCTURAL CONNECTION BETWEEN THE SLAB-ON-GRADE AND THE FOUNDATION WALL OR FOOTING.
2. THERE SHOULD BE NO CONNECTION BETWEEN THE SUBFLOOR AND PERIMETER DRAINAGE SYSTEMS.
3. THIS IS ONLY A TYPICAL BASEMENT DRAINAGE DETAIL. THE GEOTECHNICAL REPORT SHOULD BE CONSULTED FOR SITE SPECIFIC RECOMMENDATIONS.
4. THE FINAL BASEMENT DRAINAGE DESIGN SHOULD BE REVIEWED BY THE GEOTECHNICAL ENGINEER TO CONFIRM THE DESIGN IS ACCEPTABLE.

GROUNDED ENGINEERING

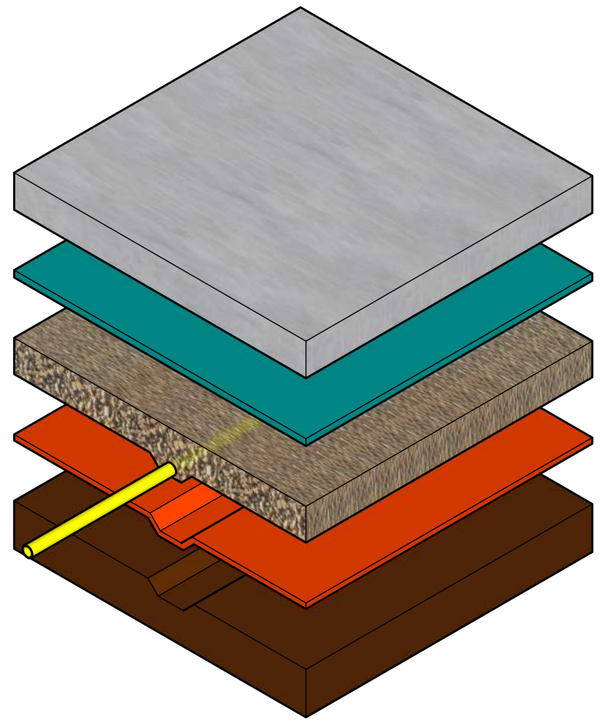
Title

FOUNDATION WALL BLINDSIDE DRAINAGE SYSTEM (IN DEEP ROCK) DETAIL

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW



ISOMETRIC VIEW

NOTES

1. WHEN THE SUBGRADE CONSISTS OF COHESIONLESS SOIL, IT MUST BE SEPARATED FROM THE SUBFLOOR DRAINAGE LAYER USING A NON-WOVEN GEOTEXTILE (WITH AN APPARENT OPENING SIZE OF < 0.250mm AND A TEAR RESISTANCE OF > 200 N).

2. TYPICAL SCHEMATIC ONLY. MUST BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT.

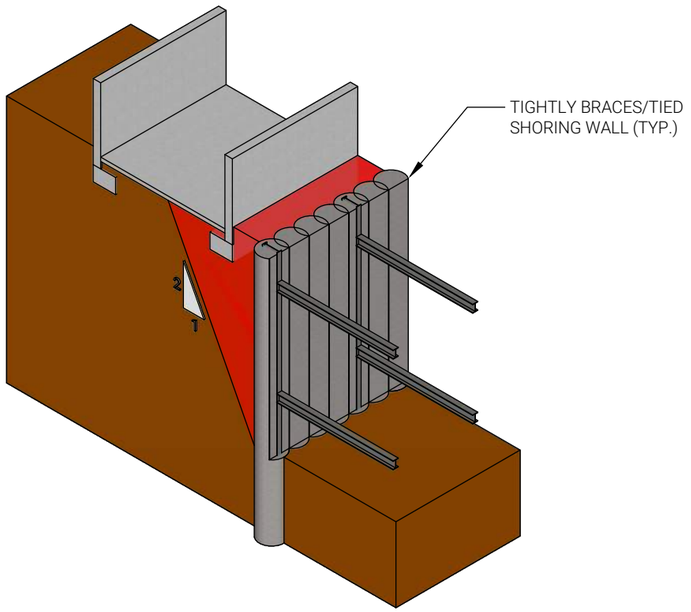
GROUNDED ENGINEERING

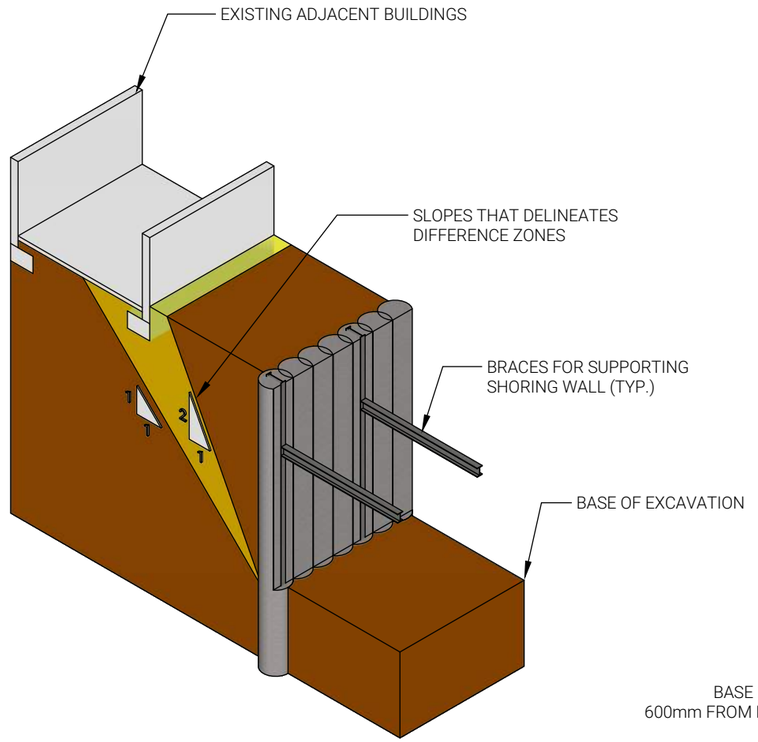
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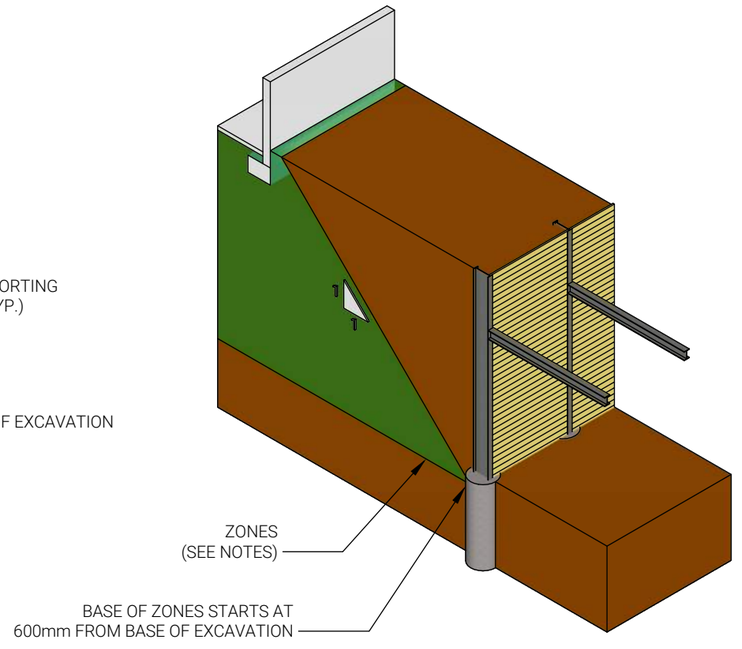
BASEMENT SUBDRAIN TYPICAL DETAIL

APPENDIX G









ZONE A (RED)

FOUNDATIONS WITHIN THIS ZONE OFTEN REQUIRE UNDERPINNING OR SHORING SYSTEM. HORIZONTAL AND VERTICAL PRESSURES ON EXCAVATION WALL OF NON-UNDERPINNED FOUNDATION MUST BE CONSIDERED

ZONE B (YELLOW)

FOUNDATIONS WITHIN THIS ZONE OFTEN DO NOT REQUIRE UNDERPINNING BUT MAY REQUIRE SHORING SYSTEM. HORIZONTAL AND VERTICAL PRESSURES ON EXCAVATION WALL OF NON-UNDERPINNED FOUNDATION MUST BE CONSIDERED

ZONE C (GREEN)

FOUNDATIONS WITHIN THIS ZONE USUALLY DO NOT REQUIRE UNDERPINNING OR SHORING SYSTEM

NOTES:
1. USER'S GUIDE - NBC 2005 STRUCTURAL COMMENTARIES (PART 4 OF DIVISION B) - COMMENTARY K.

GROUNDED ENGINEERING

Title

EXCAVATION ZONE OF INFLUENCE GUIDELINES