

GEOTECHNICAL ENGINEERING REPORT

**303 Cundles Road East,
Barrie, Ontario**

PREPARED FOR:

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Figure 1 – Site Location Plan

Figure 2 – Borehole Location Plan

Figure 3 – Subsurface Profile

Figure 4 – Subsurface Profile– with Project Units shown

APPENDICES

Appendix A – Borehole Logs; Abbreviations and Terminology

Appendix B – Geotechnical Laboratory Results

Appendix C – Chemical Analysis, Corrosivity Parameters

Appendix D – Basement Drainage Details



1 Introduction

Penady (North Barrie) Limited c/o PenEquity Realty Corporation has retained Grounded Engineering Inc. (“Grounded”) to provide geotechnical engineering design advice for their proposed development at 303 Cundles Road East, in Barrie, Ontario.

The proposed project includes constructing three (3) residential structures (Buildings B1, B2, and B3) ranging from 7 to 12 storeys. Buildings B1 and B2 have a common underground parking level set at a lowest (P1) level set at 3.6 m below Level 1. Building B3 has two underground parking levels, with the lowest (P2) level set at 6.4 m below Level 1. The Level 1 finished floor elevation (FFE) of Elev. 256.5± m was conveyed to us through email communication. The FFE for the P1 and P2 levels are calculated based on this information and the architectural drawings provided.

Building Name	Level 1 Elev. (m)	Lowest Floor Level	Lowest FFE (m)
B1 and B2	256.5	P1	252.9
B3	256.5	P2	250.1

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

- Site survey, prepared by Rudy Mak Surveying Ltd., (2016).
- Architectural Drawings, “Residential Rental / Condo, Cundles Road East, Barrie, ON”; Project S20035, dated Jan 19, 2022, prepared by SRN Architects Inc.
- Email communication, “RE: [22-014] 299 Cundles Road E, Barrie – Drilling Completed”, dated May 9, 2022, sent by Jeff Lumsden.

Grounded’s subsurface investigation of the site to date includes twelve (12) boreholes (Boreholes 1 to 12) which were advanced from February 24th to March 10th, 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 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 (NAD 83). The horizontal coordinates are provided relative to the Universal Transverse Mercator (UTM) geographic coordinate system.

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.

A summary of the relevant stratigraphic units is provided as follows. The summary elevations are provided for general guidance only. Details are provided on the borehole logs and in the following subsections. In general, four main stratigraphic units were encountered on site as follows:

1. earth fill, overlying
2. an **“upper glacial till”** unit encountered at Elev. 257.4 to 250.2± m, overlying
3. a wet **“sands and silts”** unit encountered at about at Elev. 254.5 to 245.4± m, overlying
4. a hard **“clays and silts”** unit encountered at Elev. 247.4 to 242.3± m

The groundwater varies from approximately Elev. 250.7 to 256.3± m. The groundwater is at Elev. 255.0± m at the north end of side, and Elev. 256.3± m at the south end.

2.1.1 Surficial and Earth Fill

All boreholes (except Borehole 7 and Borehole 11) encountered topsoil at ground surface, ranging in thickness between 100 and 150 mm.

Underlying the surficial materials (where observed), the boreholes observed a layer of earth fill that extends to depths of 0.8 to 4.6 metres below grade (Elev. 257.4 to 250.2 metres). The earth fill varies in composition but generally consists of silty sand to clayey silt. It contains gravel, rootlets, wood fragments, asphalt, and brick fragments. The earth fill varies in colour from brown, to brown with orange, to dark brown and black. It is moist to wet. Due to inconsistent placement and the inherent heterogeneity of earth fill materials, the relative density of the earth fill varies but is on average compact.



2.1.2 Upper Glacial Till

Underlying the fill materials, all the boreholes encountered an undisturbed native glacial till deposit. The till matrix varies widely and consists of both cohesionless sandy silt to silty sands, which are also clayey, as well as cohesive clayey silts that are also sandy. These soils are grouped together for engineering purposes as the “**upper till unit**”. This unit was encountered at 0.8 to 4.6 metres below grade (Elev. 257.4 to 250.2 m) and extends down to depths of 3.0 to 9.1 m below grade (Elev. 253.9 to 245.4 m).

The upper till is generally brown with orange or orangey brown, to grey, and moist. There are occasional wet sandy seams within the till.

Standard Penetration Test (SPT) results (N-Values) measured in the upper till range from 7 to over 61 blows per 300 mm of penetration (“bpf”), indicating a generally compact/very stiff state.

2.1.3 Sands and Silts

Underlying the upper till unit, all boreholes encountered undisturbed deposits of cohesionless sands and silts (sand, to silty sand, and sandy silt). These soils are grouped together as the “**sands and silts**” unit. This unit was encountered at 2.3 to 9.1 metres below grade (Elev. 254.5 to 245.4 m) and extends down to depths greater than 9.4 to 15.8 m below grade (Elev. 240.1 to 247.4 m) where the base of the unit was encountered. In Borehole 4, the clays and silts unit is interbedded within the sands and silts between Elev. 245.1 m and Elev. 242.2 m. This indicates that the sands and silts unit may also extend below the base of the lower clays and silts.

Boreholes 4, 5, 6, 7, 8, 9, and 10 were terminated in this unit.

The sands and silts unit is generally grey, and wet. It contains trace rock fragments and inferred cobbles.

SPT N-values measured in this unit range from 22 to 100 bpf (on average very dense, but occasionally compact).

2.1.4 Clays and Silts

Underlying the sands and silts unit, Boreholes 1, 2, 3, 4, 11, and 12 encountered a native deposit of cohesive clays and silts (clay and silt to silt and clay). These soils are grouped together as the “**clays and silts**” unit. This unit contains trace sand and light grey silt partings. This unit was encountered at 9.1 to 13.7 metres below grade (Elev. 247.4 to 242.3 m). Boreholes 1 to 3, 11, and 12 were terminated at target investigation depth in this unit.

The clays and silts are generally grey and moist. SPT N-values measured in this unit range from 22 to 86 bpf (on average hard, but occasionally very stiff).



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. The boreholes were cased by hollow stem augers on completion, and cave measurement was not practical. Monitoring wells were installed in select boreholes, and stabilized groundwater levels were measured in each of the monitoring wells one month after the completion of drilling.

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

Borehole No.	Borehole depth (m)	Upon completion of drilling		Strata Screened	Water Level in Well, Depth/Elev. (m)	
		Depth to cave (m)	Unstabilized water level (m)		Apr 12, 2022	May 6, 2022
1	14.3	n/a	filled with drill water	Clays and Silts (Elev. 245.3 to 242.3± m)	4.0 / 252.0	4.0 / 252
2	15.8	n/a	filled with drill water	Sands and silts (Elev. 245.3 to 242.3± m)	3.8 / 250.7	3.7 / 250.8
4	15.8	n/a	filled with drill water	Sands and silts (Elev. 242.2 to 240.6± m)	0.9 / 255.0	0.9 / 255.0
6	11.2	n/a	filled with drill water	Sands and silts (Elev. 248.0 to 244.9± m)	1.0 / 254.6	0.8 / 254.8
8	11.1	n/a	filled with drill water	Sands and silts (Elev. 250.5 to 247.5± m)	2.5 / 255.7	2.5 / 255.7
10	11.3	n/a	filled with drill water	Sands and silts (Elev. 248.6 to 245.6± m)	1.3 / 254.9	1.3 / 254.9
11	15.8	n/a	filled with drill water	Clays and Silts (Elev. 244.8 to 243.3± m)	2.3 / 256.2	2.2 / 256.3
12	11.3	n/a	filled with drill water	Sands and silts (Elev. 250.7 to 247.7± m)	1.9 / 254.9	1.7 / 255.1

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.

The design groundwater table for engineering purposes is at Elev. 256.3± m in the south portion of the site (for footprint of buildings B1 and B2), and Elev. 255.0± m in the north portion of the site (for building B3). The sands and silts unit has high permeability and will yield free-flowing water when penetrated. There is also infiltrated stormwater perched in the earth fill.

Grounded has prepared a hydrogeological report for this site (File No. 22-014).

2.3 Corrosivity and Sulphate Attack

Five (5) 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	BH1-SS8	BH4-SS6	BH5-SS6	BH6-SS5	BH11-SS5
Soluble Sulphate (SO ₄) in soil sample	81 µg/g < 0.1 %	71 µg/g < 0.1 %	< 20 µg/g < 0.1 %	29 µg/g < 0.1 %	< 20 µg/g < 0.1 %
Class of Exposure	Negligible	Negligible	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									
	BH1-SS8		BH4-SS6		BH5-SS6		BH6-SS5		BH11-SS5	
	Result	Points	Result	Points	Result	Points	Result	Points	Result	Points
Resistivity (ohm.cm)	5610	0	5350	0	8010	0	3530	0	7800	0
pH	8.04	0	7.97	0	8.00	0	8.01	0	7.95	0
Redox Potential (mV)	265	0	286	0	272	0	283	0	292	0
Sulfides (%)	0.000037	2	0.000022	2	0.000058	2	0.000032	2	0.000042	2
Moisture (%)	8.8	2	10.6	2	10.1	2	9.62	2	9.21	2
Corrosion protection recommended?	No		No		No		No		No	
Resistivity less than 2000 ohm.cm?	No		No		No		No		No	

The analytical results only provide an indication of the potential for corrosion. All five 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 five 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 topsoil and earth fill soils are considered unsuitable for the support of the proposed building foundations.

The lowest FFE for each building has been calculated based on the assumed ground floor elevations and are shown in the following table. The buildings B1 and B2 have a common lowest P1 level that is connected. Building B3 has the lowest underground level at P2 level.

Building Name	Applicable Boreholes	Borehole Ground Elevations (m)	Assumed Ground Floor Elev. (m)	Lowest Floor Level	Assumed Lowest FFE (m)	Groundwater Table Elev. (m)
B1 and B2	BH6 to BH12	255.6 to 258.5	256.5	P1	252.9	256.3
B3	BH1 to BH5	254.5 to 256.0	256.5	P2	250.1	255.0

3.1.1 Spread Footings

Foundations made for the proposed **P1 level for Buildings B1 and B2** will bear on undisturbed (dewatered) dense/hard native soils. Conventional spread footings made to bear on this soil may be designed using a maximum factored geotechnical resistance at ULS of 500 kPa. The net geotechnical reaction at SLS is 350 kPa, for an estimated total settlement of 25 mm. Individual spread footing foundations must be at least 2000 mm wide and must be embedded a minimum of 1000 mm below FFE. If higher capacities are required, foundations may also be made as cast-in place spread footings or drilled piers to bear at Elev. 250.0± m in the undisturbed dense / very dense sands and silts. Conventional spread footings made as drilled piers bearing on very dense sands and silts at Elev. 250± m may be designed using a maximum factored geotechnical resistance at ULS of 1,200 kPa. The net geotechnical reaction at SLS is 800 kPa, for an estimated total settlement of 25 mm.



Foundations made for the proposed **P2 level** for building B3 will bear on undisturbed stiff/compact native subgrade. Conventional spread footings made to bear on this soil may be designed using a maximum factored geotechnical resistance at ULS of 350 kPa. The net geotechnical reaction at SLS is 250 kPa, for an estimated total settlement of 25 mm. Individual spread footing foundations must be at least 2000 mm wide and must be embedded a minimum of 1000 mm below FFE.

These minimum requirements apply 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.

Prior to excavation, it will be necessary to positively dewater for any excavations. These excavations must be dewatered to a minimum 1.2 m below the deepest proposed excavation elevation prior to excavation, to preserve the in-situ integrity of the native soils. If the subsurface is not dewatered prior to excavation, the native soils will become disturbed by the ingress of groundwater and the above recommendations for bearing capacity will not be valid.

When exposed to ambient environmental temperatures, the design earth cover for frost protection of foundations and grade beams is 1.5 metres. The lowest levels of unheated underground parking structures two or more levels deep are, although unheated, still warmer than typical outdoor winter temperatures. 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 (such as in a P1 parking garage), 1.5 m of earth cover (or equivalent insulation) is required for frost protection.

Footings stepped from one elevation to another should be offset at a slope not steeper than 7 vertical to 10 horizontal.

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.

3.1.2 Alternative Foundation Options

If the structural engineer deems that the spread footing capacities provided above are not sufficient to carry the proposed column loads, the following foundation options are provided as an alternative to spread footings..

3.1.2.1 CFA Piles

Continuous flight auger piles (“CFA Piles”), which are also known as auger pressure grouted piles can be used for high-capacity foundations and can provide both shaft resistance and toe



resistance. This deep foundation approach will avoid the issue of water pressure disturbing the bases of the caissons as soon as the augers are removed from the boring (regardless of whether the shaft is lined or not).

CFA piles are installed using hollow stem continuous flight augers. When the required depth is reached, high strength fluid grout is pumped under pressure through the hollow stem augers exiting through the tip. A pre-designed grout volume is pumped under pressure prior to lifting the auger to build up a “grout head” around the outside of the auger. The auger is then withdrawn in a controlled manner slowly rotating clockwise as the grout pumping continues to both maintain the head of grout and avoid any intrusion of water or soil into the grout column.

CFA piles are designed based on the combination of tip resistance and skin friction. For design purposes, the longer the CFA piles, the greater the resistance provided.

The factored geotechnical resistance at ULS is provided on the basis of a geotechnical resistance factor of 0.4 as per the Canadian Foundation Engineering Manual and the Ontario Building Code.

CFA piles spaced apart at greater than 4 diameters (measured centre to centre) will not induce group effects. Piles placed closer than this will induce group effects, and a reduced bearing capacity will apply, which is dependent on caisson sizing, bearing stratum, founding elevation, and separation distance. The individual pile capacity will be reduced if multiple piles (pile group) are to be installed. The pile layout and details must be reviewed by Grounded to assess the pile group efficiency/capacity. A detailed review of the design building loads and pile capacity will have to be conducted for the design for this option.

A static load test (ASTM D1143 - Standard Test Methods for Deep Foundations Under Static Axial Compressive Load) must be performed at this site to confirm or improve the above capacities. An Osterberg Cell Load Test is also feasible on CFA piles. The factored geotechnical resistance at ULS provided above uses a factor of 0.4. When the static load test is performed the reduction factor will be 0.6, which will most likely provide a higher factored geotechnical resistance at ULS. The load test will also provide settlement parameters. It is anticipated that the geotechnical reaction at SLS will be similar to the ULS value.

3.1.2.2 Raft Foundation

If long-term groundwater extraction / discharge is not preferred/permitted, then basement structures must be made fully watertight. Further, the spread footing capacities for the P2 level may not be sufficient for the support of the proposed B3 structure (although they may be sufficient for the support of the podium/low rise portions).

A raft foundation could be designed to carry structural loads, with waterproofed foundation walls designed to withstand hydrostatic forces (lateral and uplift).

Buildings B1 and B3 are irregular in shape. Based on the tower footprint, equivalent rectangles were used as the raft dimensions for bearing capacity discussions below. Raft slabs for a podium structure will be subjected to much less load, and will not govern design.



Building	Raft Area Assumed	Proposed Underground Structure	Foundation Approach	Preliminary Capacities
Building B1	32 m x 69 m 2180 m ²	P1 (Elev. 252.9 m)	Raft (founded at 251.4± m)	fULS: 2,000 kPa SLS: 90 kPa @ 25 mm 130 kPa @ 50 mm MSR: 1,800 kPa/m
Building B2	22 m x 65 m 1420 m ²	P1 (Elev. 252.9 m)	Raft (founded at 251.4± m)	fULS: 1,700 kPa SLS: 110 kPa @ 25 mm 170 kPa @ 50 mm MSR: 2,600 kPa/m
Building B3	36 m x 56 m 1880 m ²	P2 (Elev. 250.1 m)	Raft (founded at 248.6± m)	fULS: 2,000 kPa SLS: 120 kPa @ 25 mm 160 kPa @ 50 mm MSR: 1,600 kPa/m

The preliminary raft design parameters are provided assuming a *uniform load* at the base of the raft for the assumed raft area/sizes provided in the table above. 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 are provided as the initial step in determining raft feasibility (a structural task).

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.

The raft design parameters are based on conservatively assessed Young's Moduli (virgin and unload-reload) for each of the load-bearing strata, and can likely be improved by additional in situ testing of the Young's Modulus (pressuremeter testing) within the critical portions of the zone of influence of the raft, in future boreholes.

Settlement parameters can likewise 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 MSR's under the raft are then sent back to the structural engineer, and the structural design is modified as necessary.

Should raft slab design be considered beyond this conceptual level, recommendations and discussion pertaining to differential settlement between the podium and tower must be provided by Grounded. This may involve detailed construction sequencing or the use of delay strips.

During construction, the subgrade at founding elevation should be cut neat, inspected, and immediately protected by a mud slab (comprising lean concrete) to provide a working surface. The subsurface must not be proofrolled as this activity would further weaken these soils. The raft slab is then constructed on top of the mud slab. Prior to pouring the mud mat and 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.

If the entire basement is to be fully waterproofed, 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. 256.3± m is to be used for excavations pertaining to buildings B1 / B2 and a design groundwater table of Elev. 255.0± m is to be used for excavations pertaining to building B3.

Differential settlement is related to real non-uniform raft load distribution and must be assessed as part of the detailed design process. Differential settlement will become an issue if two different foundation types (conventional spread footings and deep foundations) are used to support structures with different column loads (e.g. towers and adjacent podiums) on a shared underground parking structure. Net geotechnical reactions at SLS have been provided for both foundations systems, which will occur as load is applied and is linear and non-recoverable. The tolerance for differential settlement is related to the structural design and is specified by the structural engineer as a function of column spacing. To avoid this issue, any combination of scheduling, delay strips, and increasing raft spacing may be employed.

Tiedown Anchors

If deemed necessary by the structural engineer, micropile or helical pier tiedowns can be designed to resist uplift. In the very dense subgrade below founding elevation, it is expected that post-grouted micropile anchors can be made such that an anchor will safely carry up to 70 kN/m in tension at ULS of adhered anchor length (at a nominal diameter of 150 mm). Helical pile anchors made in the very dense/hard subgrade (Chance SS225 elements assumed) can be designed as well. 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.

Helical pile detailed design will ultimately depend upon the loading considerations and the ground conditions. The project geotechnical information should be provided to a specialist design/build contractor to assess the feasibility of this foundation system and to determine probable helical pile refusal/installation depths. The actual installation depth of each helical pile is determined on site during installation based on depth and torque measurements made during installation, and the load support requirements. The load carrying capacity of each helical pile is confirmed by the helical pile contractor based on the torque measurements and a full-scale performance test of a



prototype/production pile. Occasionally, field torque measurements indicate that helical piles must be advanced deeper than originally designed. Provision must be made in helical pile contracts to allocate and quantify risks associated with any extra time and materials utilized to achieve the required field torque readings. The presence of cobbles or boulders in the native soil (to be anticipated) will impede helical pile installation. Refer to the borehole logs for detailed subsurface information. Provision must be made in helical pile contracts to allocate risks associated with the time spent and equipment utilized to remove or work around such obstructions when encountered.

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 of 250.9-248.1± metres, the boreholes observe dense to very dense sands below the proposed buildings B1, and B2; and observe very stiff to hard tills and clays and silts below the proposed building B3. The cohesive tills and the clays and silts have an average undrained shear strength greater than 100 kPa. Based on this information, the site designation for seismic analysis is **Class C**, 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
Upper Till	21	34	0.28	0.44	3.54
Sands and Silts	21	36	0.26	0.41	3.85
Clays and Silts	21	34	0.28	0.44	3.54



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 ($\gamma - 9.8$ 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 the basements are to be fully waterproofed, the full height of the basement walls should be watertight and designed to withstand horizontal hydrostatic pressure below Elev. 256.3± m for the south portion of the site (Buildings B1 and B2) and Elev. 255.0± m for the north portion of the site (Building B3).

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 soil subgrade and the base of the footing. The factored geotechnical resistance to friction (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 Slab on Grade Design Parameters

The slab-on-grade parameters provided here apply to a conventional slab on grade and drained basement approach only. The structure may be fully watertight and designed to withstand uplift and hydrostatic pressures, with no permanent drainage. In this case, conventional slab-on-grade design parameters do not apply. Design parameters for a raft foundation are provided in Section 3.1.



At the proposed lowest FFEs (P1 or P2), the undisturbed (dewatered) native soils will provide adequate subgrade for the support of a conventional slab on grade. The modulus of subgrade reaction for slab-on-grade design supported by undisturbed native soils is 30,000 kPa/m.

If this basement structure is made as a conventional drained structure, a permanent drainage system including subfloor drains is required (see Section 3.5). In this case, the slab on grade must be provided with a drainage layer and capillary moisture break. For **Building B3**, this is to be achieved by forming the slab on a minimum 300 mm thick layer of HL8 coarse aggregate (OPSS.MUNI 1150) or HPB vibrated to a dense state.

For **Buildings B1 and B2**, as slab-on-grade is to be made on cohesionless wet sands, the drainage layer must be separated from the sands using a non-woven geotextile and Granular A filter. In this case, the bulk excavation is subexcavated a minimum 150 mm and replaced by Granular A (OPSS.MUNI 1010) compacted to 98% of SPMDD. A non-woven geotextile (with an apparent opening size of less than 0.250 mm and a tear resistance of more than 200 N) is to be placed on the surface of the compacted Granular A. The subfloor drains are then laid directly on the flat subgrade and backfilled with a minimum 300 mm thick layer of HL8 coarse aggregate (OPSS.MUNI 1150) or HPB, vibrated to a dense state. Any solid collection pipes must be sloped so that they positively discharge to the sumps.

Without proper filtering there may be entry of fines from the surrounding subgrade soils into the bedding. This loss of ground could result in a loss of support of the slab and clogging of the subfloor drainage system.

Given the nature of the soils at this site, recompaction or proof rolling of the undisturbed subgrade will weaken the subgrade materials. These activities should be specifically prohibited when preparing the subgrade. The subgrade should be cut neat and inspected by Grounded prior to placement of the capillary moisture break and construction of the slab. Disturbed or otherwise unacceptable material (as determined by Grounded) must be subexcavated and replaced with Granular B (OPSS.MUNI 1010) compacted to a minimum of 98% SPMDD. 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.

3.5 Long-Term Groundwater and Seepage Control

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 3 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. How the drainage system is installed depends on whether the basement wall is made in an open cut or over a shored excavation face. 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.

In an open cut excavation, basement wall drainage is installed directly against the basement wall from the open cut side. Perimeter foundation drains made in this application comprise perforated pipe (minimum 100 mm diameter) surrounded by a granular filter of OPSS.MUNI HL-8 Coarse Aggregate providing a minimum 300 mm of cover over the drain pipe.

Typical basement drainage 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.

If for any reason this basement is to be fully watertight, the structure should be 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. 256.3± m for the south portion of the site (Buildings B1 and B2) and Elev. 255.0± m for the north portion of the site (Building B3).

The permanent dewatering requirements are provided in Grounded's Hydrogeological Report (File No. 22-014).

3.6 Site Servicing

All services must have at least 1.5 metres of earth cover or equivalent insulation for frost protection.

Where site services are not installed below the basement levels of the proposed development, the following recommendations apply.



3.6.1 Bedding

The soil subgrade encountered within utility trenches on site may consist of either earth fill or native soil. If earth fill is encountered, the subgrade must be compacted in place to a minimum 98% SPMDD. The trench base must be inspected for obvious loose, wet, or disturbed material. Any unsuitable material must be subexcavated and replaced with imported fill compacted to 98% SPMDD.

As trenches will extend below the groundwater table, the groundwater table must be lowered to 1.2 m below the lowest excavation elevation prior to excavation.

Bedding material must consist of well graded granular fill such as Granular A (OPSS.MUNI 1010). Clear stone is specifically prohibited at this site. The bedding material must be compacted to a minimum 95% SPMDD.

3.6.2 Backfill

Excavated earth fill and native soils on site will constitute adequate backfill material if the soil meets the backfill specifications:

- Any deleterious material in the earth fill is removed prior to reuse as backfill.
- The moisture content is within 2% of optimum, or moisture conditioned to within 2% of optimum.
- The backfill must be compacted to a minimum 98% SPMDD.

3.6.3 Trench Plugs

Trench plugs are installed when the invert of the trench is below the groundwater table, to prevent the groundwater from preferentially flowing through the granular bedding and backfill material, creating a local drawdown of the groundwater table. Where local drawdown is not tolerated, trench plugs can be installed in the granular bedding and backfill material. Trench plugs may be constructed as clay plugs or cut off collars around the pipe barrel.

Clay plugs should be installed every 50 m along the full length of the trench, where the trench invert is below the groundwater table. Clay plugs must be a minimum of 1 m thick along the length of the trench and will completely replace any bedding or backfill material around the pipe barrel. Material used for clay plugs must have greater than 15% of the particles finer than 2 microns and a coefficient of permeability of less than 10^{-8} m/s. The material must be compacted to 95% SPMDD. Unshrinkable fill is also a suitable clay plug material. A representative sample of clay plug material must be submitted prior to construction and during construction for permeability and particle size testing to confirm the material is adequate and in compliance with the above material specifications.

If cut off collars are used instead of clay plugs, the cut off collar must not be placed within 1 m of a pipe joint to ensure adequate compaction. The soils around the cut off collar must be



compacted to 95% SPMDD. A watertight connection is required between the collar and the pipe wall.

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 upper till is a Type 3 soil
- The wet sands are Type 4 soils, or Type 3 soils if dewatered

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.

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.

4.2 Short-Term Groundwater Control

Considerations pertaining to groundwater discharge quantities and quality are discussed in Grounded's hydrogeological report for the site, under separate cover.



Positive dewatering to lower the groundwater table prior to excavation will be required to facilitate construction as well as to maintain the integrity of the subgrade for foundation and slab-on-grade support. Dewatering will take some time to accomplish prior to the start of excavation. The water level must be kept at least 1.2 m below the lowest excavation elevation during construction. Failure to dewater prior to excavation will result in unrecoverable disturbance of the subgrade, which will render advice provided for undisturbed subgrade conditions inapplicable.

If a watertight basement is 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. 256.3± m for the south portion of the site (Buildings B1 and B2) and Elev. 255.0± m for the north portion of the site (Building B3) is to be used in this assessment.

It is recommended that a professional dewatering contractor be consulted to review the subsurface conditions and to design a site-specific dewatering system. It is the dewatering contractor's responsibility to assess the factual data and to provide recommendations on dewatering system requirements.

Should the excavation be supported using permeable soldier pile and lagging shoring, positive dewatering will be required on a continuous ongoing basis during excavation and throughout construction.

To better control groundwater during construction for Buildings B1 and B2, a fully continuous interlocking concrete caisson wall may be considered as one potential approach to shoring the excavation. A caisson wall could be designed as a coffer dam with a 3-5 m toe (piles and fillers), which would reduce steady state groundwater volumes, or alternatively it could be made as a proper cut-off wall toed into the lower clays and silts unit. If a full caisson cut-off wall is preferred, additional boreholes are required to delineate the cutoff elevation, which is highly variable at this site and was not observed in Boreholes 5 to 10.

A properly constructed full shoring cut-off wall approach is advantageous as it will provide a fully-continuous temporary groundwater cut-off barrier (i.e. piles and fillers), which will enable the site to be dewatered during construction without inducing more horizontal flow into the excavation. Dewatering inside an excavation protected by a full cut-off barrier wall may be conducted using conventional sump arrangements. Once the inside of the cut-off wall excavation is dewatered, precipitation events will be the primary contributor of water entering the excavation is shored with a cut-off wall.

4.3 Earth-Retention Shoring Systems

No excavation shall extend below the foundations of existing adjacent structures without adequate alternative support being provided. Excavation zone of influence 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 with active dewatering prior to and during construction.

The caisson wall approach should be considered for Buildings B1 and B2 as it will provide a fully-continuous temporary groundwater cut-off barrier (i.e. piles and fillers), which will enable the site to be dewatered during construction without inducing more lateral flow into the excavation. In this case, caisson wall shoring comprises the entire shoring system.

Since the boreholes surrounding the proposed Buildings B3 show cohesive tills and silts and clays, a soldier pile and lagging system may be considered for this excavation.

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:

$$P = 0.8 K[\gamma H + q] + \gamma_w h_w \dots \text{in cohesive soils}$$
$$P = 0.65 K[\gamma H + q] + \gamma_w h_w \dots \text{in cohesionless soils}$$

- 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. 256.3± m for the south portion of the site (Buildings B1 and B2) and Elev. 255.0± m for the north portion of the site (Building B3) must be accounted for. There is infiltrated stormwater perched in the earth fill and upper till which may accumulate behind a caisson wall. This hydrostatic pressure needs to be accounted for in shoring design.

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. In cohesionless soils, the lateral earth pressure distribution is rectangular.



4.3.2 Soldier Pile Toe Embedment

Soldier pile toes will be made in undisturbed (dewatered) wet sands and silts, or hard clays and silts units. Soldier pile toes resist horizontal movement due to the passive earth pressure acting on the toe below the base of excavation.

There are zones of soil in the subgrade that are wet, cohesionless, and permeable. Augered holes for piles made into these soils will be prone to caving and blowback. Temporarily cased holes are required to prevent borehole caving during installations in drilled holes. To prevent groundwater issues (groundwater inflow, caving and blowback into the drill holes, disturbance to placed concrete, etc.) during drilling and installation, construction methods such as utilizing temporary liners, pre-advancing liners deeper than the augured holes, mud/slurry/polymer drilling techniques, or other methods as deemed necessary by the shoring contractor are required. Tremie placement of concrete may be required if there is more than 50 mm of water at the bottom of a caisson prior to placement of concrete.

The piezometric head in the sands and silts could cause basal heave and groundwater inflow to the drill hole. To protect drill holes against basal disturbance caused by the inflow of groundwater, and to prevent loss of ground, it will be necessary at this site to control the bases of any drill holes during drilling. This may include dewatering to below the drill hole depths prior to installation, or the use of drilling muds (slurry, polymer, etc.), pre-advancing casing, or other techniques as deemed necessary by the shoring contractor.

4.3.3 Lateral Bracing Elements

The shoring system at this site will require lateral bracing. 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.

In the very dense sands below, it is expected that post-grouted anchors can be made such that an anchor made in the sands and silts unit will safely carry up to 70 kN/m of adhered anchor length (at a nominal borehole diameter of 150 mm). Post-grouted anchors made in the upper till unit and the clays and silts unit will safely carry up to 50 kN/m of adhered anchor length (at a nominal borehole diameter of 150 mm).

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 very dense sands below the proposed FFE is suitable for the placement of raker foundations. For Buildings B1 and B2, raker footings established on very dense soils at an inclination of



45 degrees can be designed for a maximum factored geotechnical resistance at ULS of 300 kPa. For Building B3, raker footings established on very stiff to hard clays can be designed for a maximum factored geotechnical resistance at ULS of 150 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.

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

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.

This geotechnical engineering feasibility study is intended for due diligence purposes only. At detailed design, site-specific boreholes, groundwater monitoring wells, and updated detailed geotechnical engineering advice are required. Once completed, the future detailed geotechnical engineering report by Grounded Engineering would then supersede this preliminary report.

5.1 Investigation Procedures

The geotechnical engineering analysis and advice provided 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. 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 Penady (North Barrie) Limited c/o PenEquity Realty Corporation 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 local municipal/regional governing bodies may also make use of and rely upon this report, subject to the limitations as stated.

6 Closure

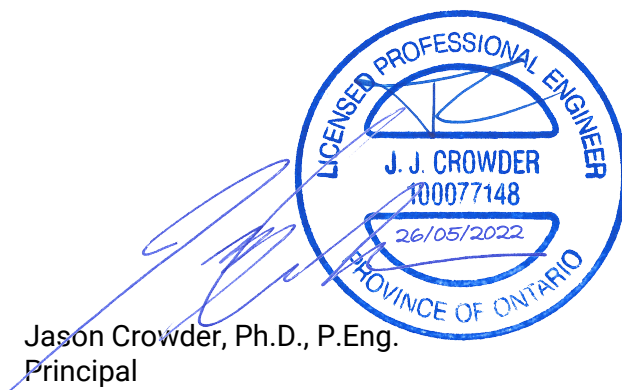
If the design team has any questions regarding the discussion and advice provided, please do not hesitate to have them contact our office. We trust that this report meets your requirements at present.

For and on behalf of our team,



Deepak Kanraj, M.A.Sc, EIT
Project Manager

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



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

FIGURES





GROUND
ENGINEERING

1 BANIGAN DRIVE, TORONTO, ONT., M4H 1G3
www.groundedeng.ca

LEGEND

— APPROXIMATE PROPERTY BOUNDARY

Note

Reference

ArcGIS Map 2022

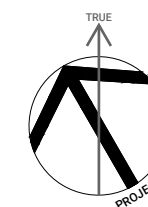
Project

**303 CUNDLES RD E
BARRIE ONTARIO**

Figure Title

SITE LOCATION PLAN

North



Date

APRIL 2022

Scale

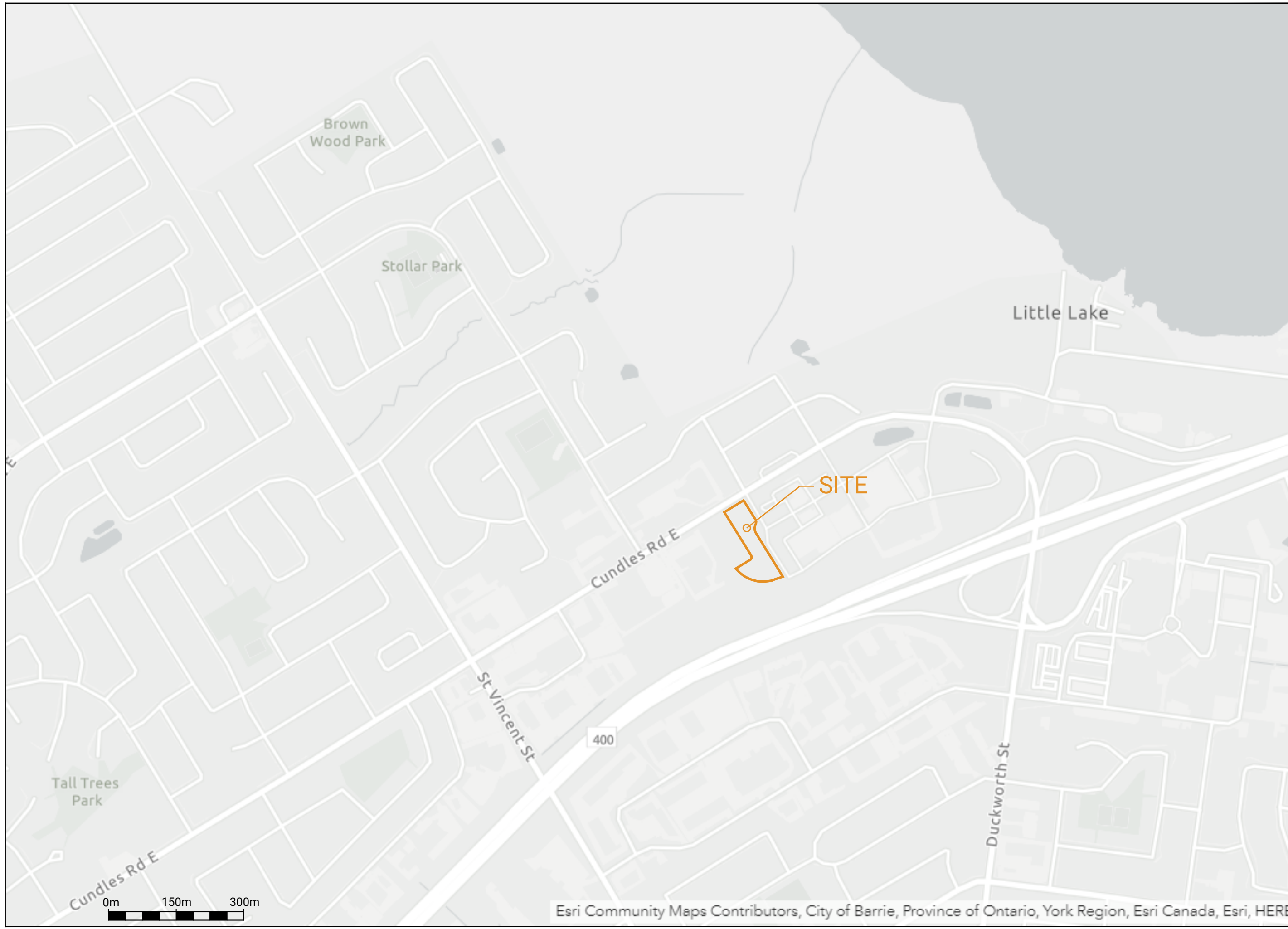
AS INDICATED

Job No

22-014

Figure No

FIGURE 1





GROUND
ENGINEERING

1 BANIGAN DRIVE, TORONTO, ONT., M4H 1G3
www.groundedeng.ca

LEGEND

- APPROXIMATE PROPERTY BOUNDARY
- ⊕ MONITORING WELL/BOREHOLE BY GROUNDED

Note

Reference

ArcGIS Map 2022

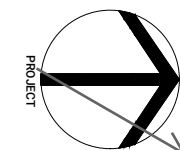
Project

**303 CUNDLES RD E
BARRIE ONTARIO**

Figure Title

SITE LOCATION PLAN

North



Date

APRIL 2022

Scale

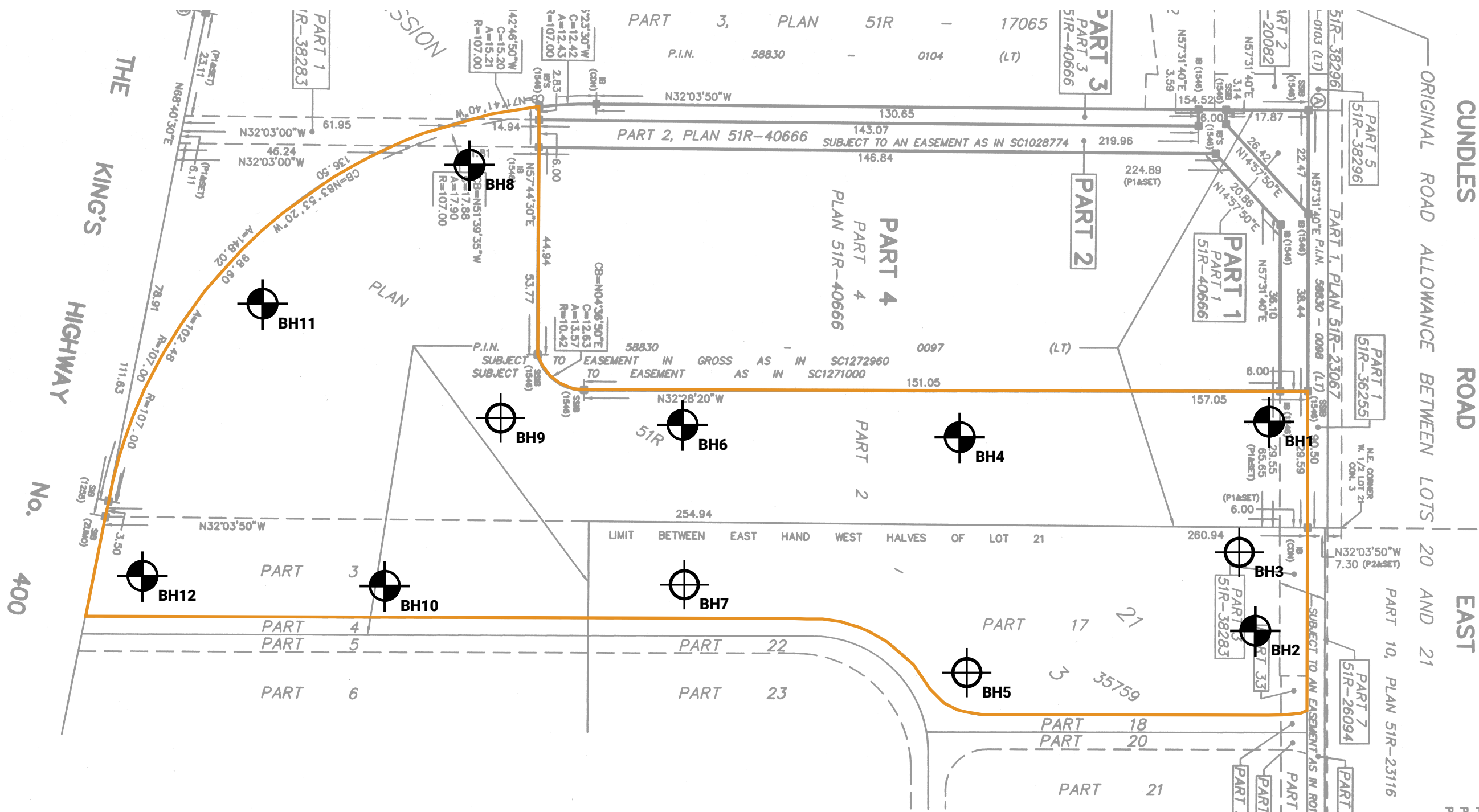
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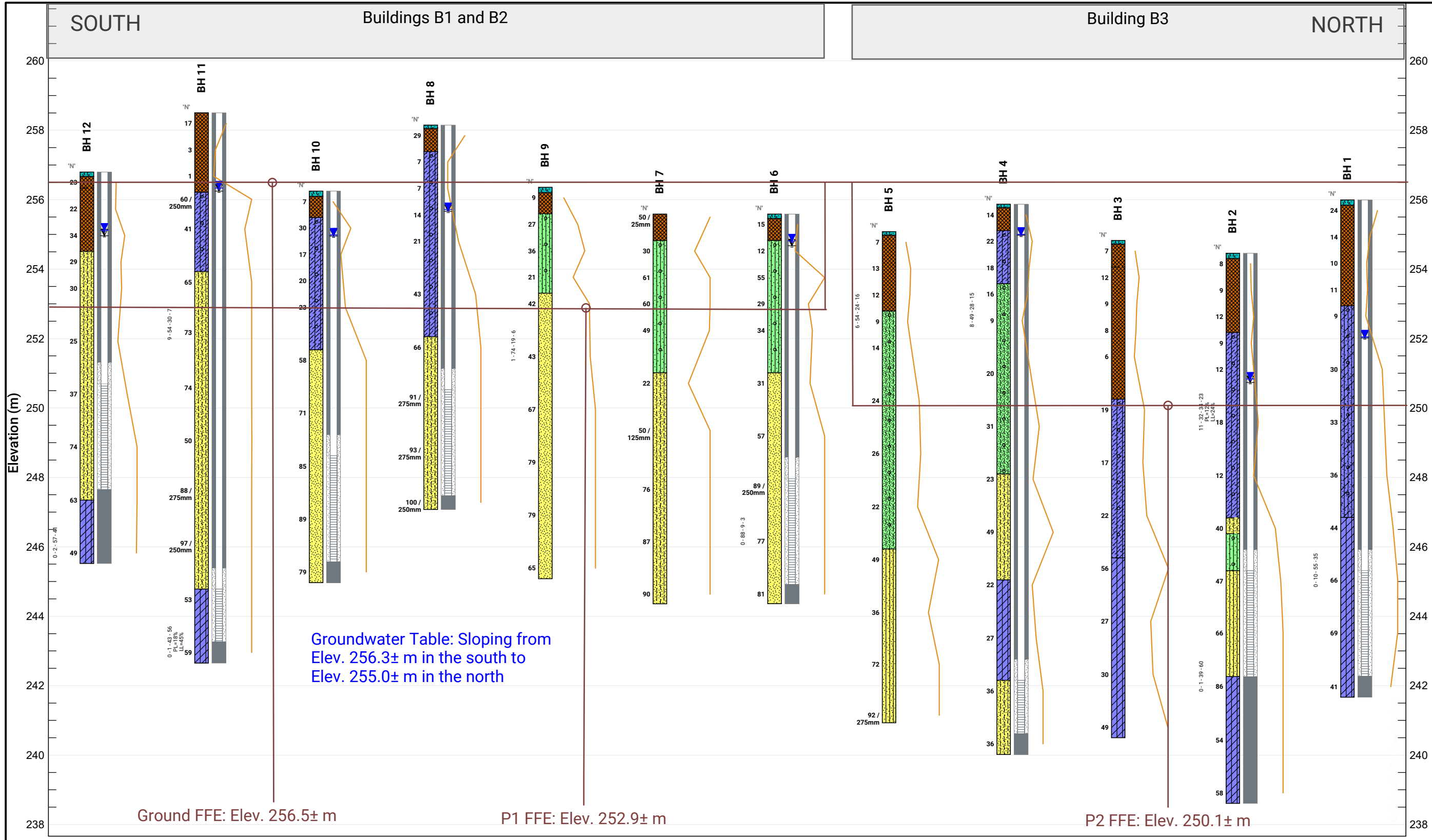
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22-014

Figure No

FIGURE 2





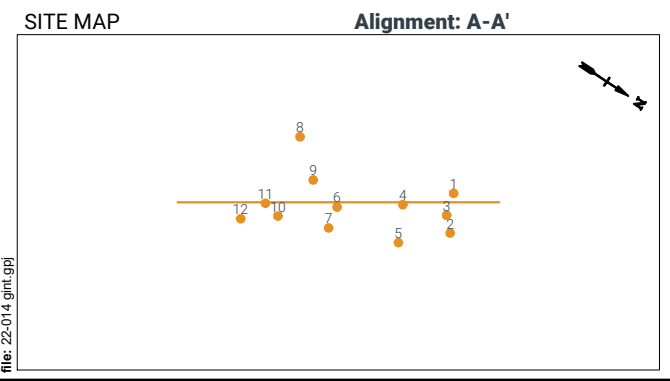
LEGEND

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

water level, unstabilized
 water level, stabilized (latest)
 water level, stabilized (highest)

Project
**303 CUNDLES RD E
BARRIE, ON**

Figure Title
**SUBSURFACE PROFILE
A-A'**



BOREHOLE STRATIGRAPHY LEGEND

Boreholes Equally Spaced

Date	MAY 2022
Scale	AS INDICATED
Job No	22-014
Figure No	FIGURE 3

APPENDIX A



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
 unstabalized 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
silty	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 mm² 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.

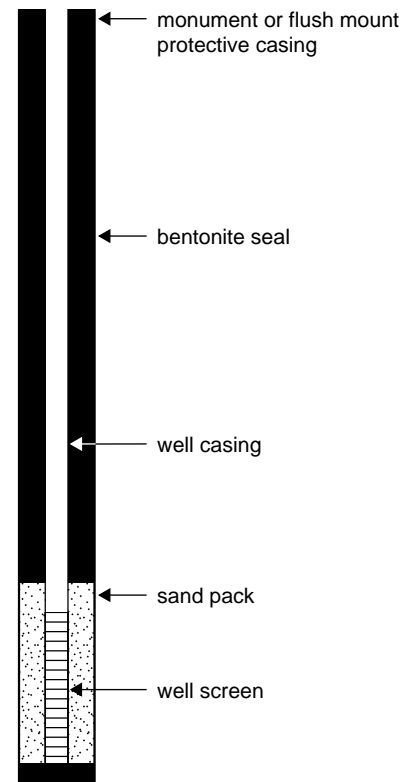
ASTM D1587 Shelby Tubes (ST)

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.

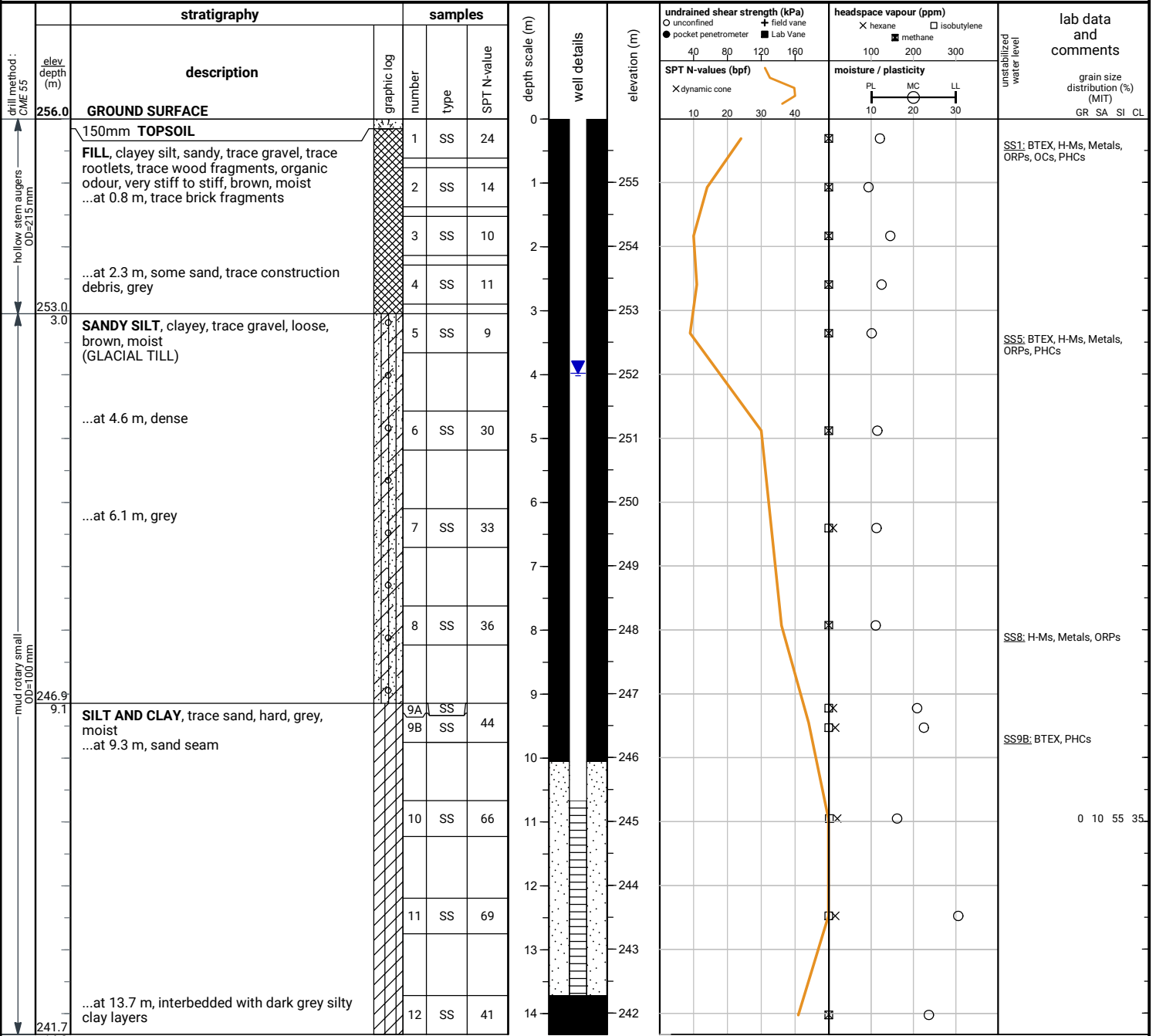
ASTM D4719 Pressuremeter Test (PMT)

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



File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation

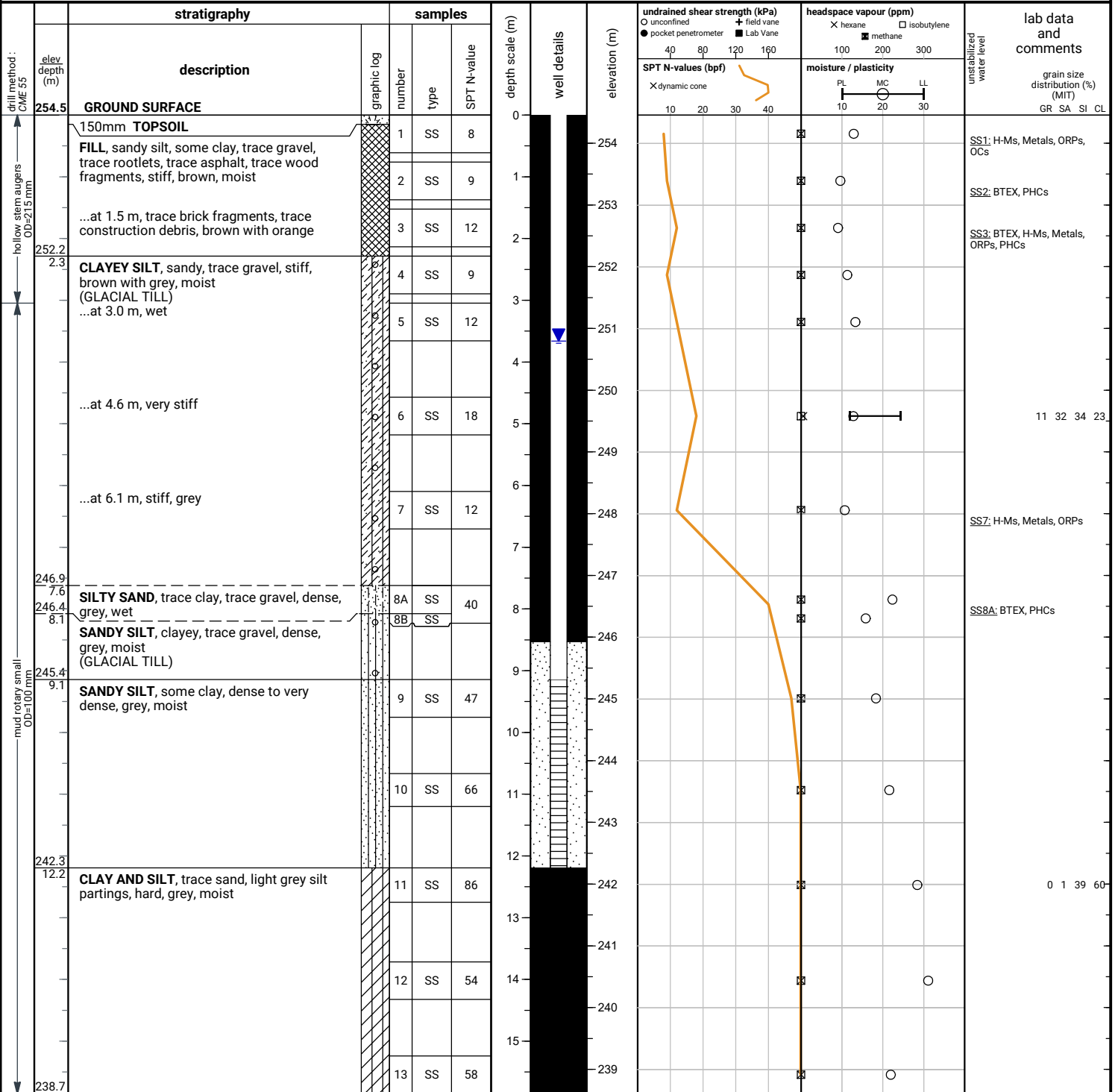


END OF BOREHOLE

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)
Apr 12, 2022	4.0	252.0
May 6, 2022	4.0	252.0

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

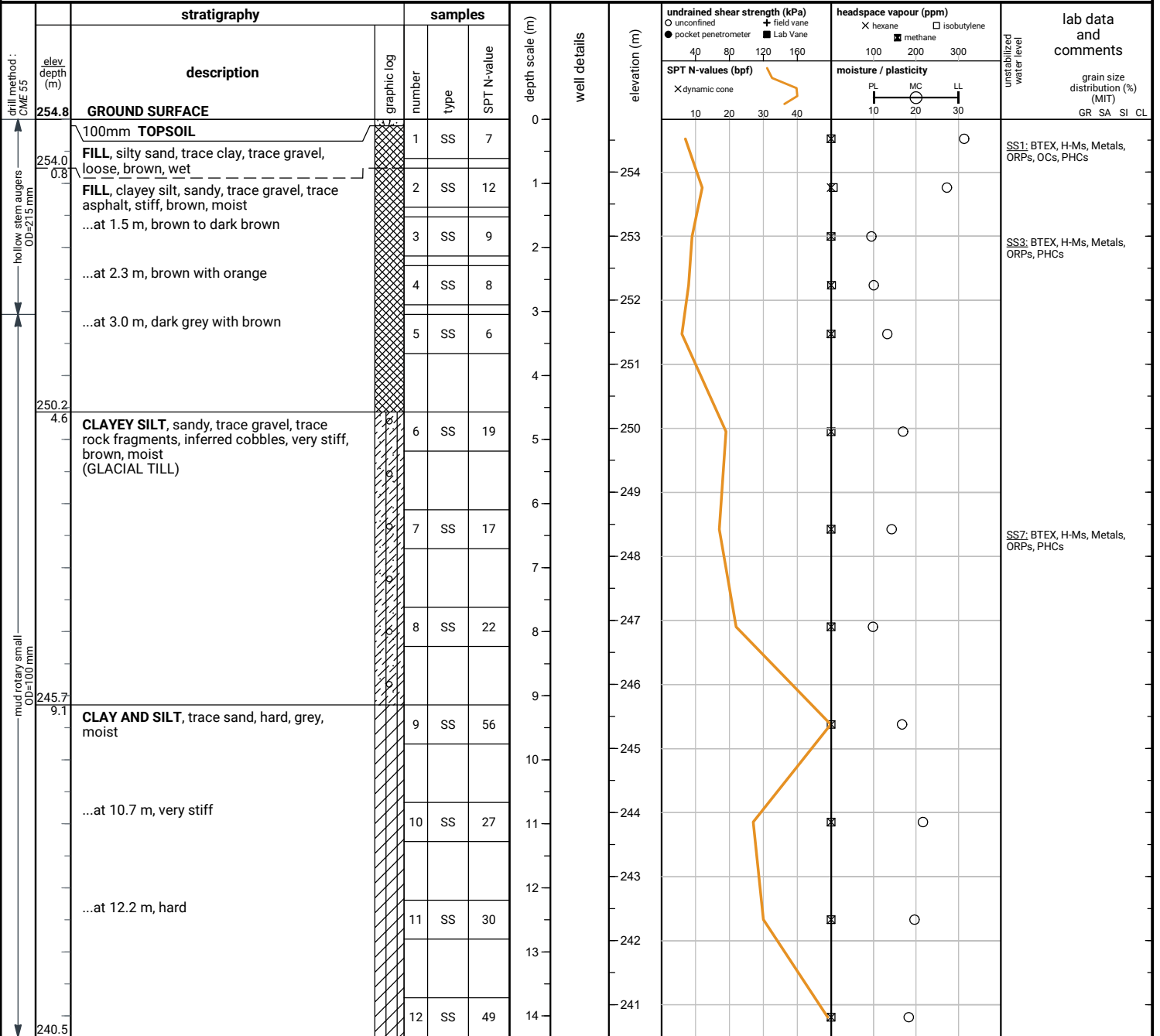
Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	3.8	250.7
May 6, 2022	3.7	250.8

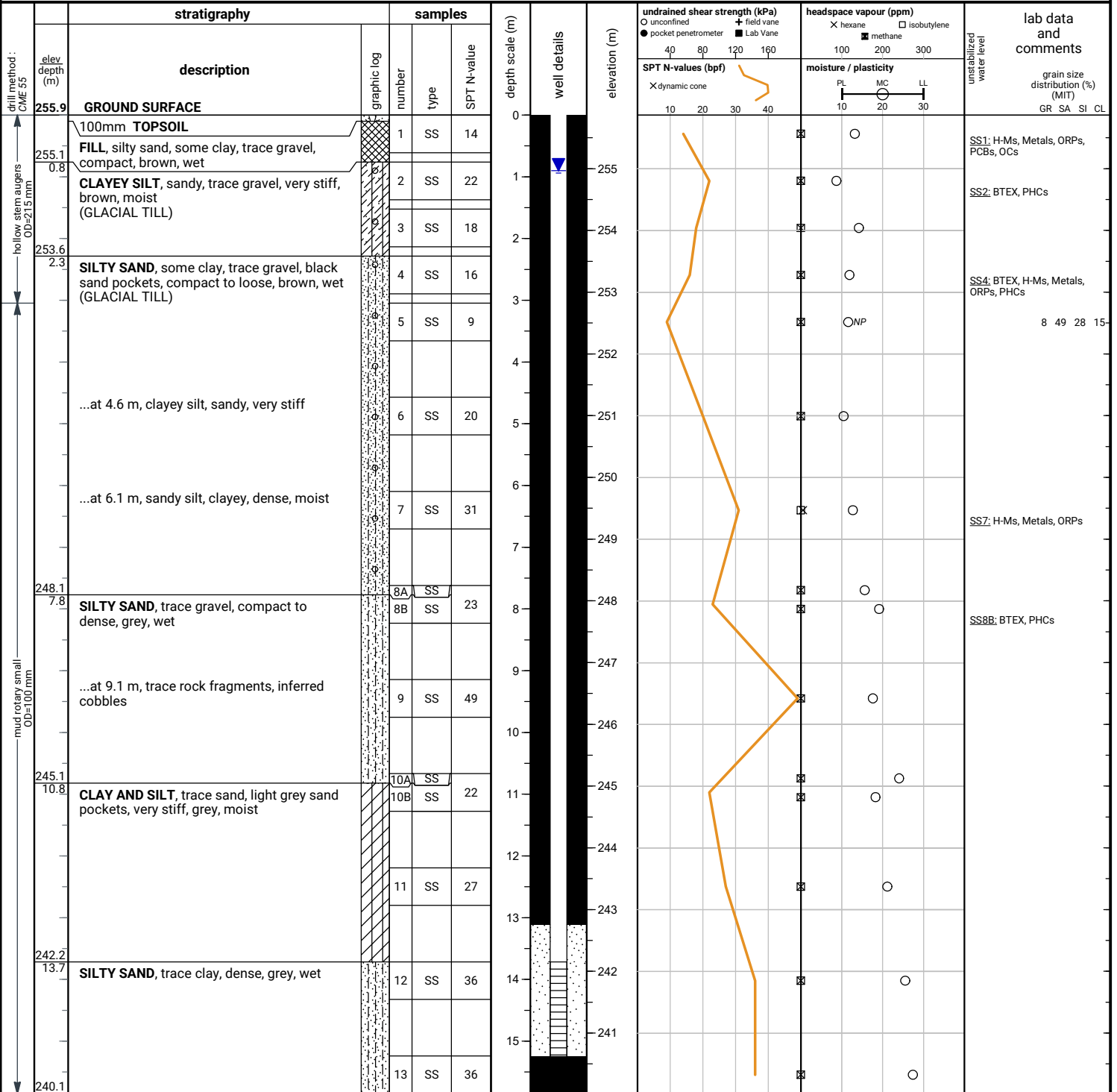
File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

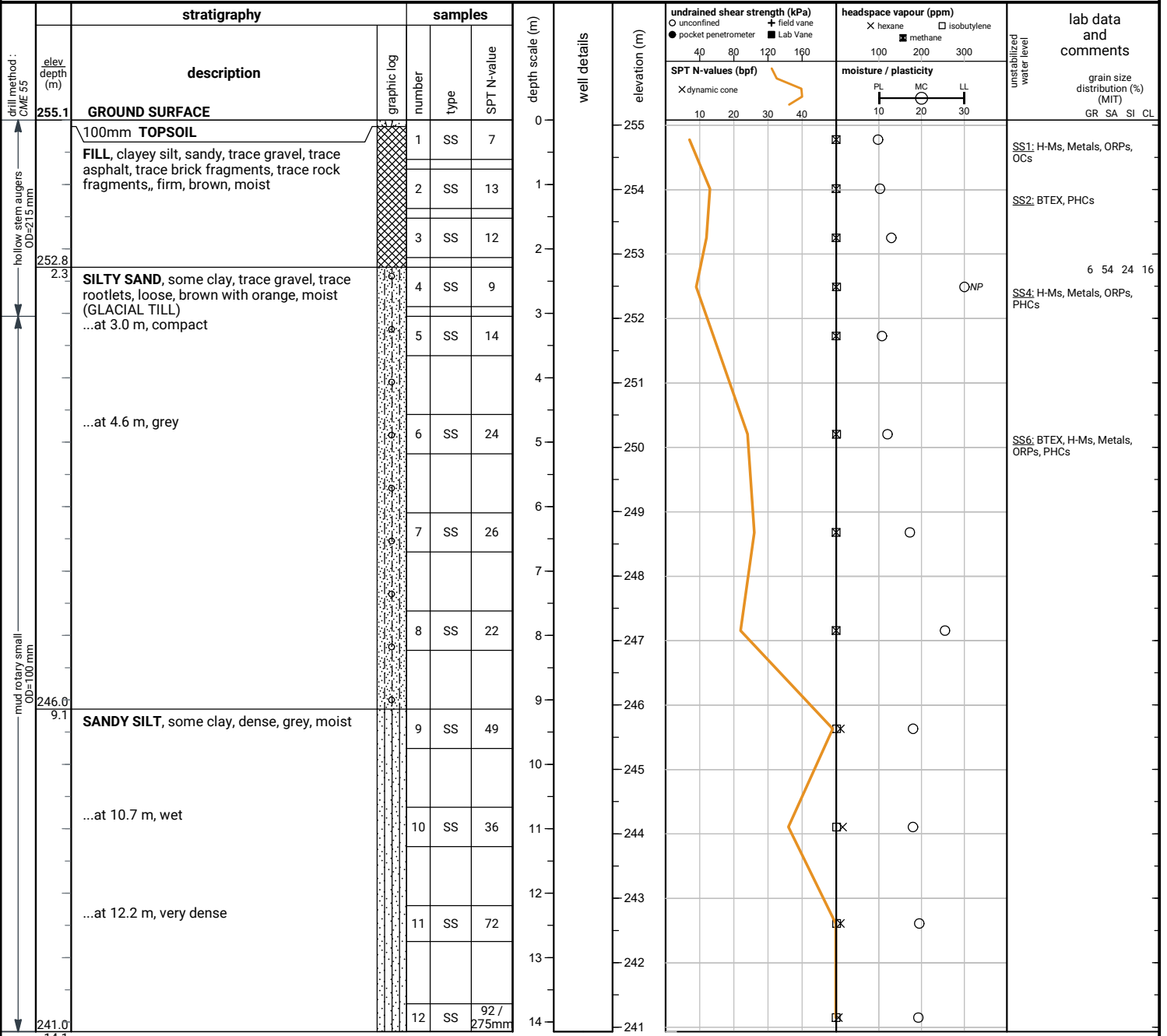
Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	0.9	255.0
May 6, 2022	0.9	255.0

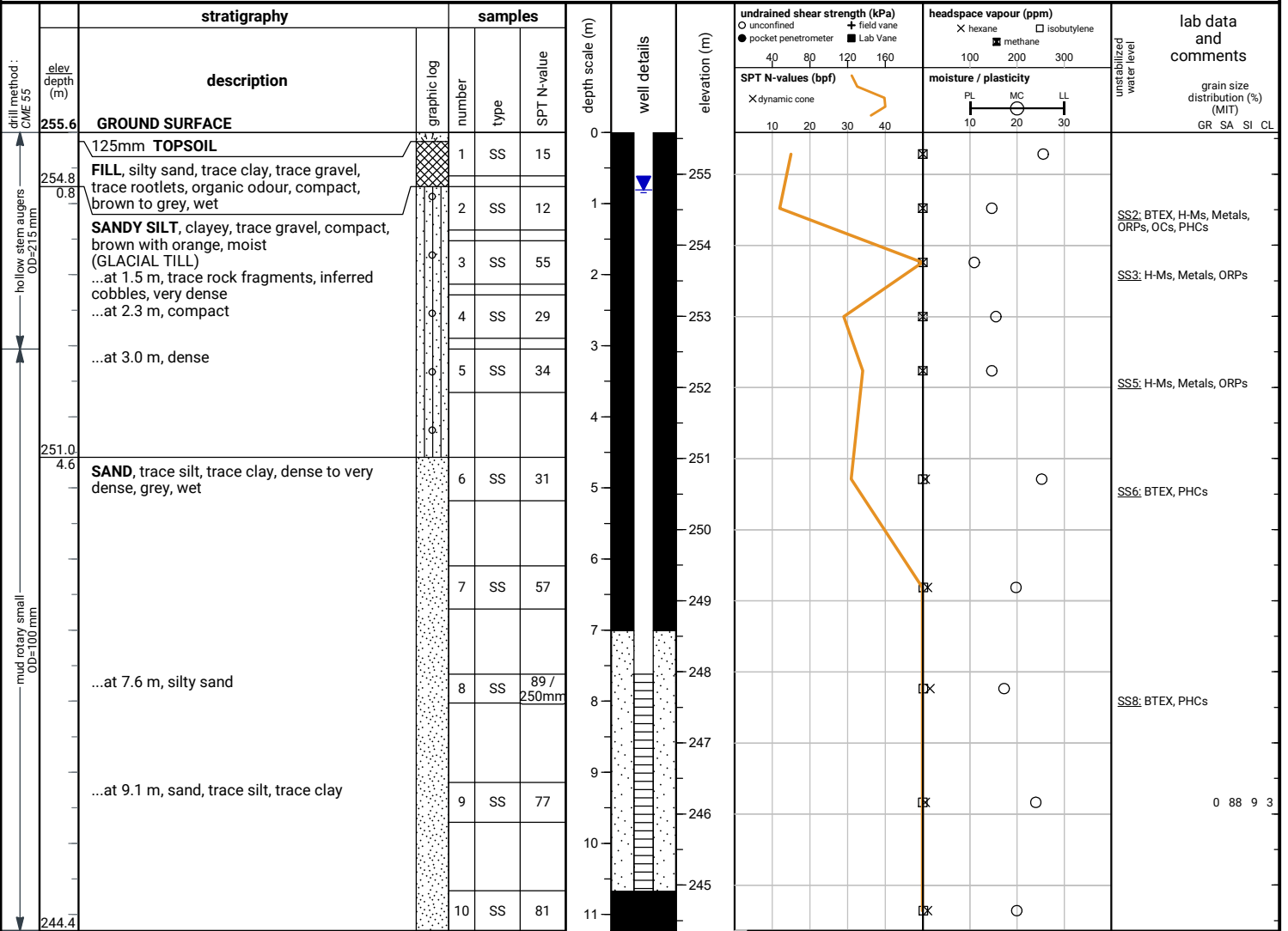
File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

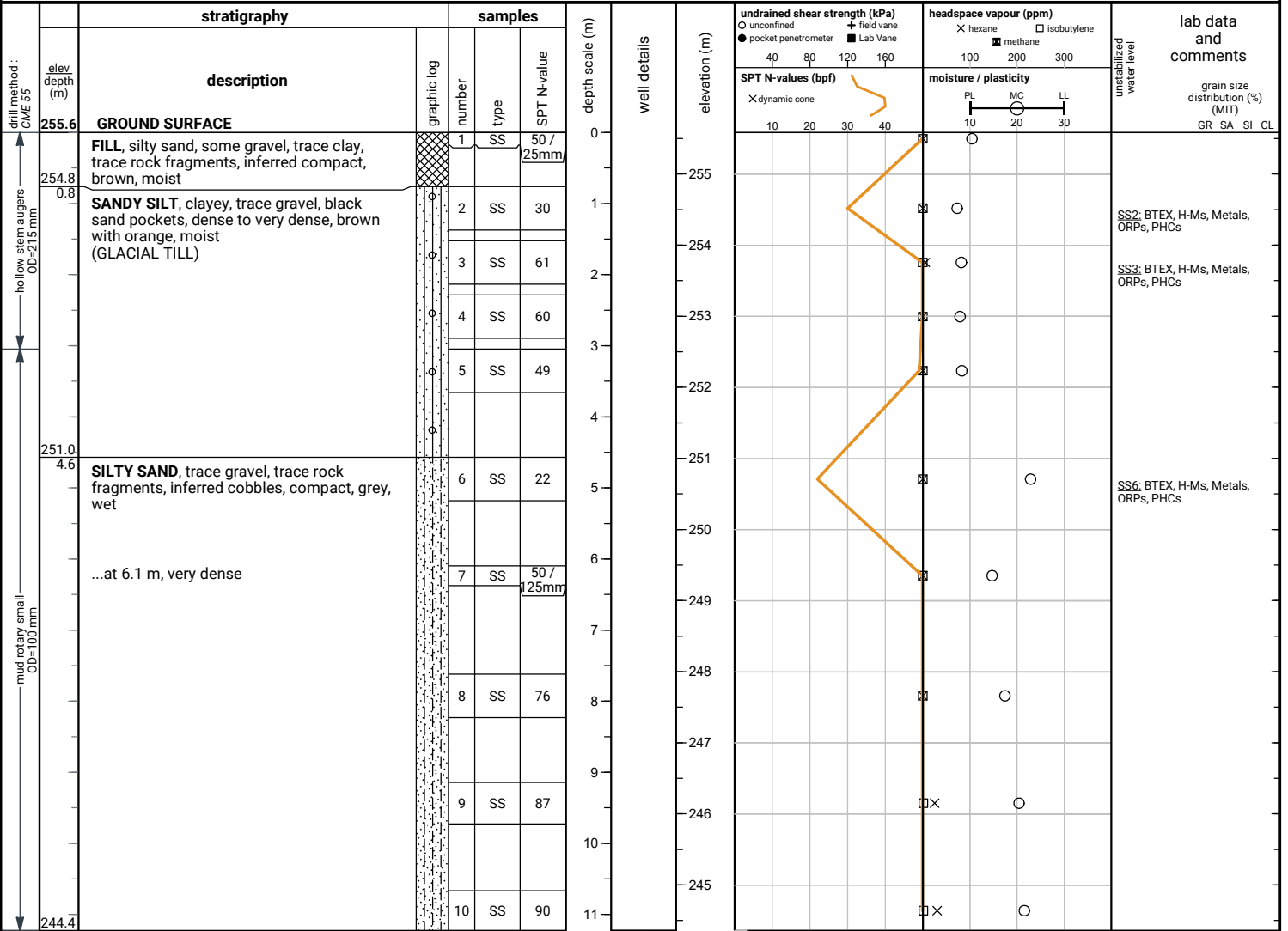
Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	1.0	254.6
May 6, 2022	0.8	254.8

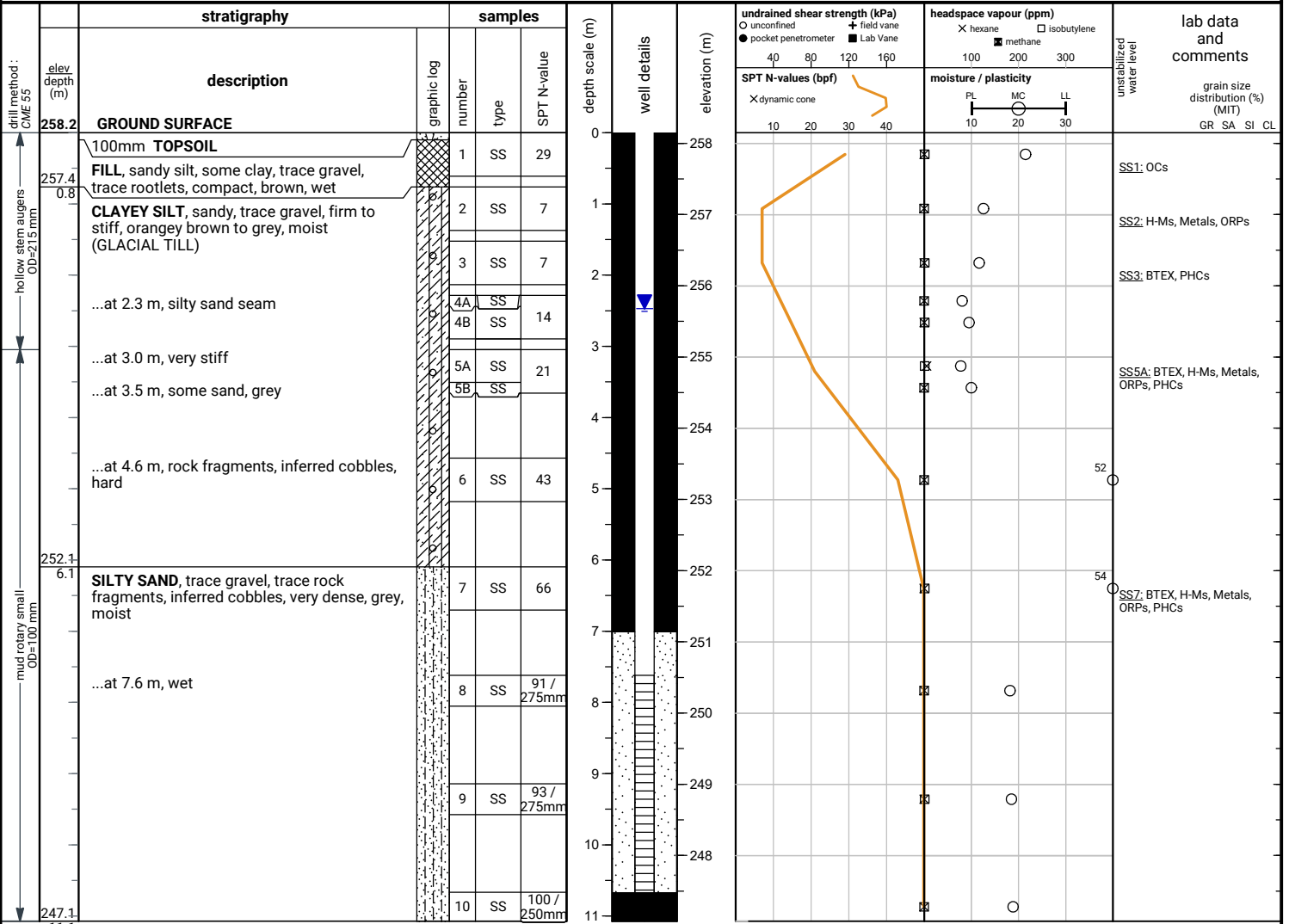
File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

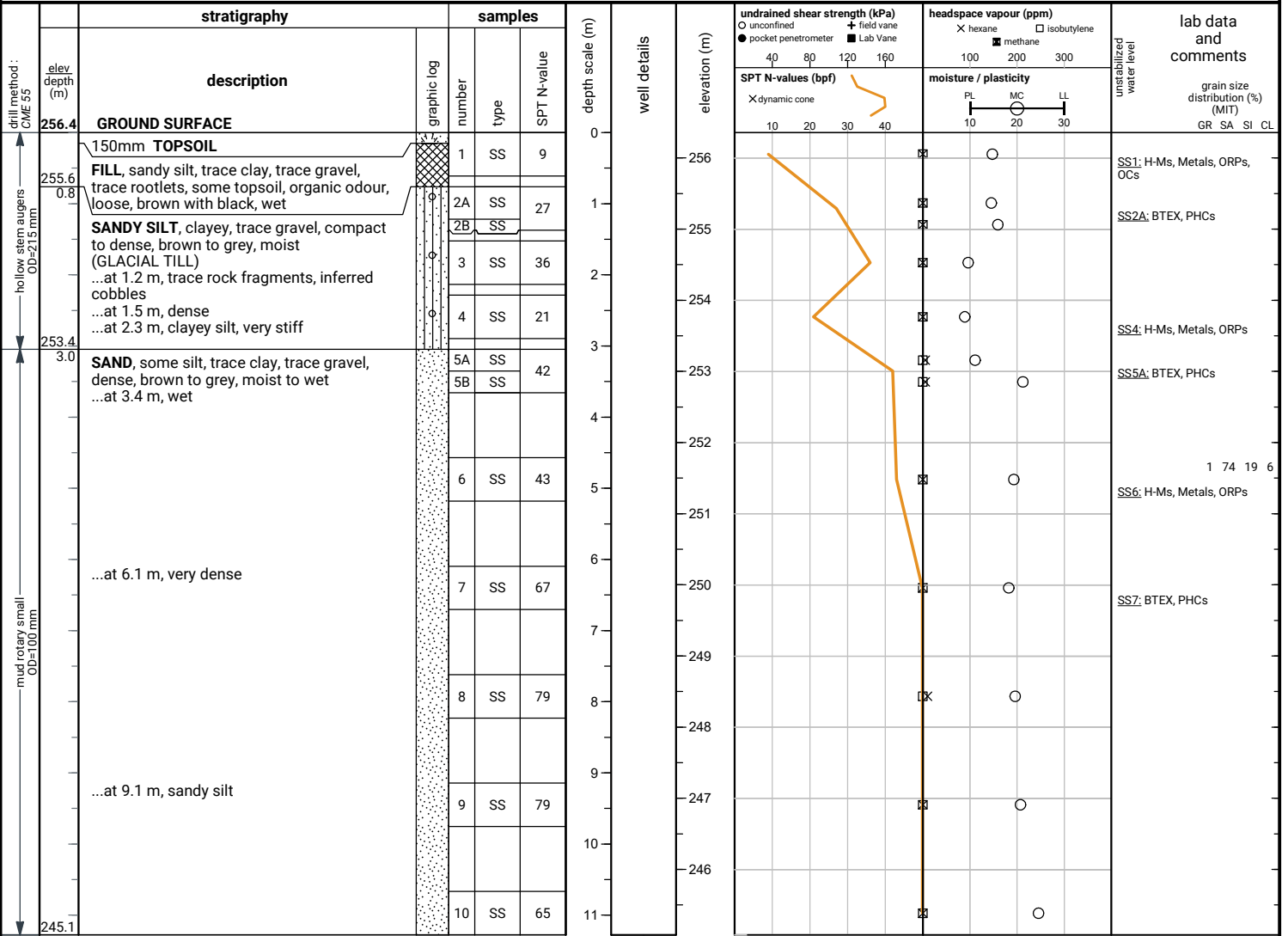
Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	2.5	255.7
May 6, 2022	2.5	255.7

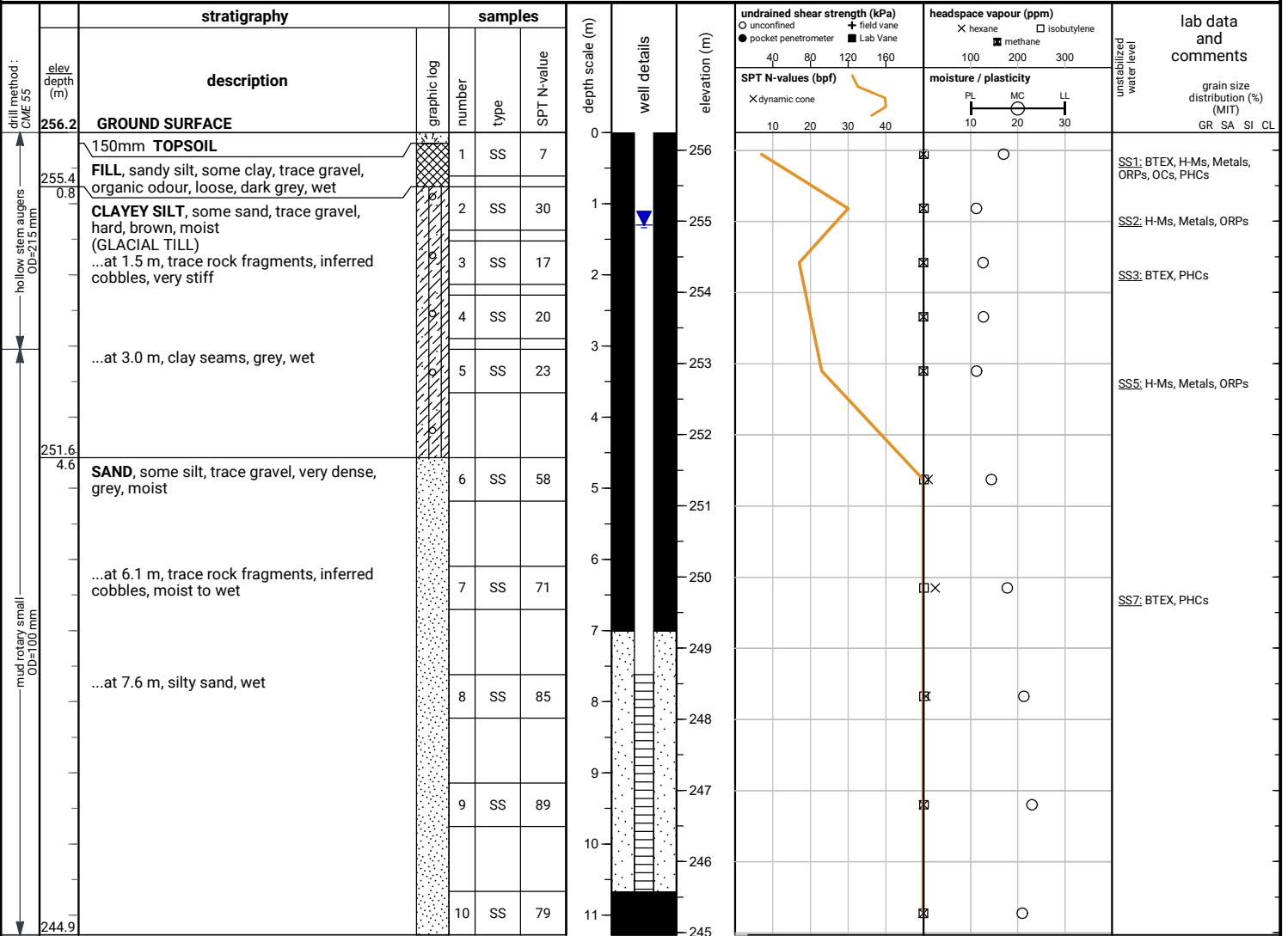
File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

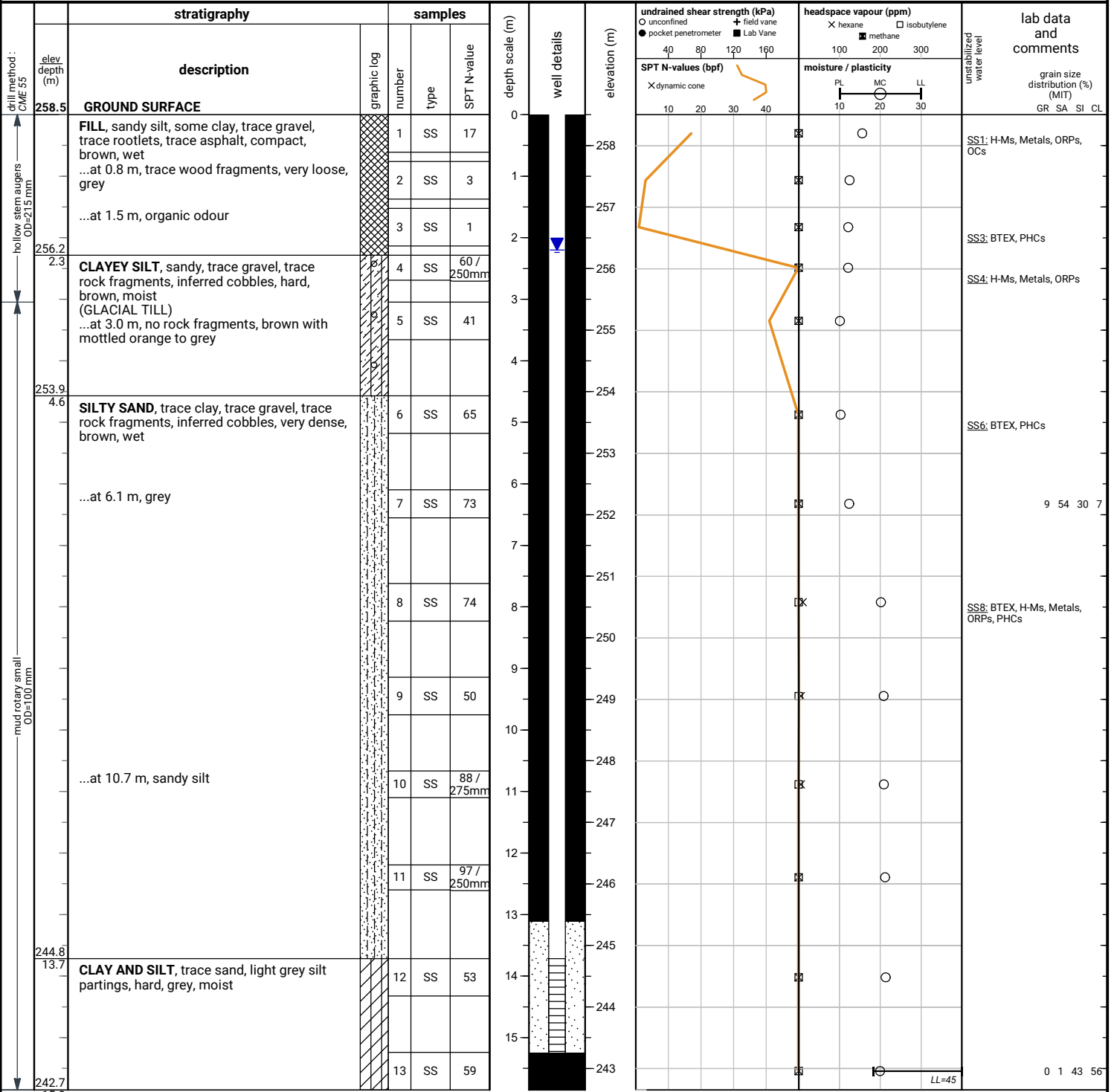
Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	1.3	254.9
May 6, 2022	1.3	254.9

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

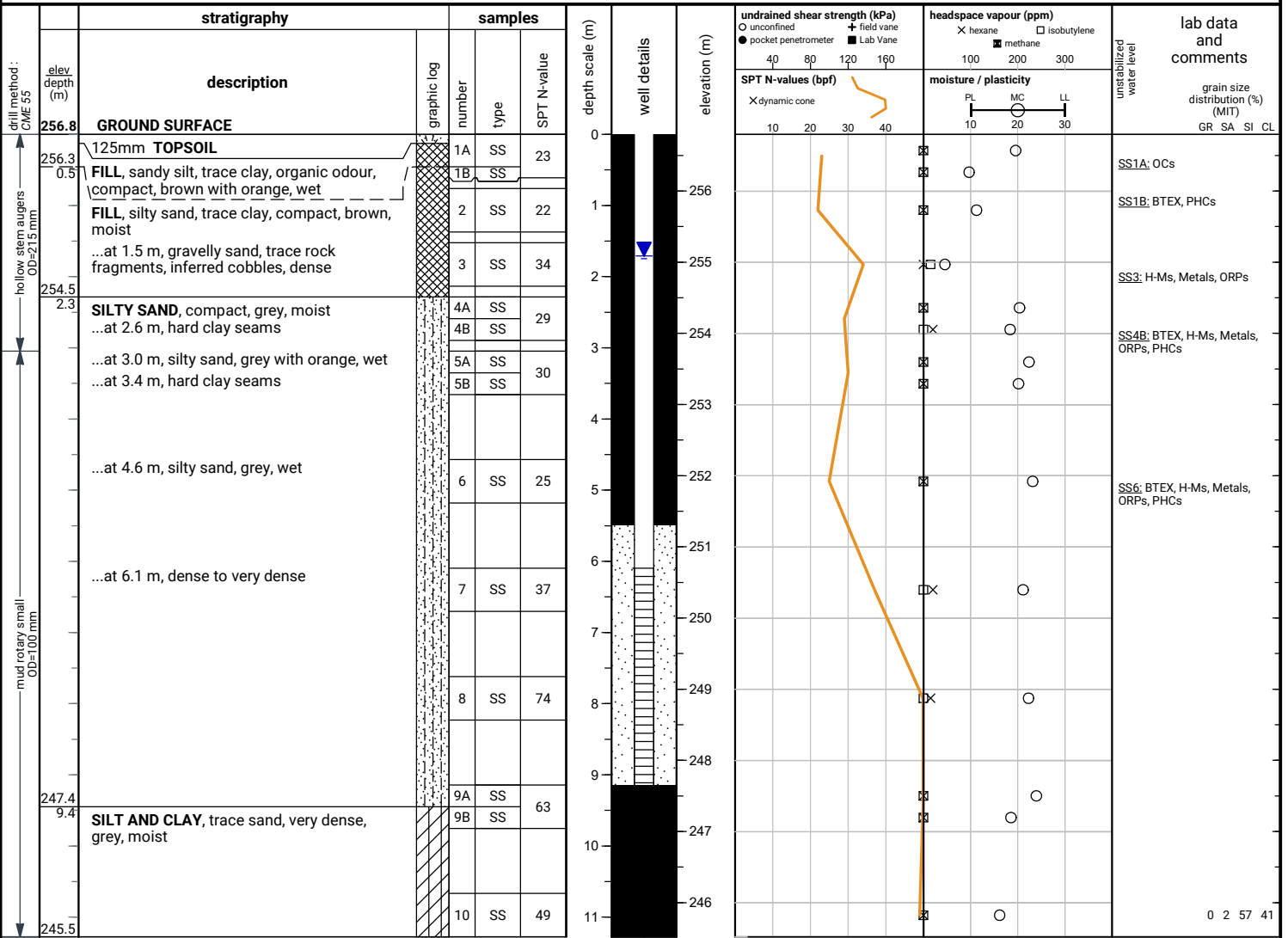
Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	2.3	256.2
May 6, 2022	2.2	256.3

File No. : 22-014 Project : 303 Cundles Rd E, Barrie, ON Client : Penady (North Barrie) Limited c/o PenEquity Realty Corporation



END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

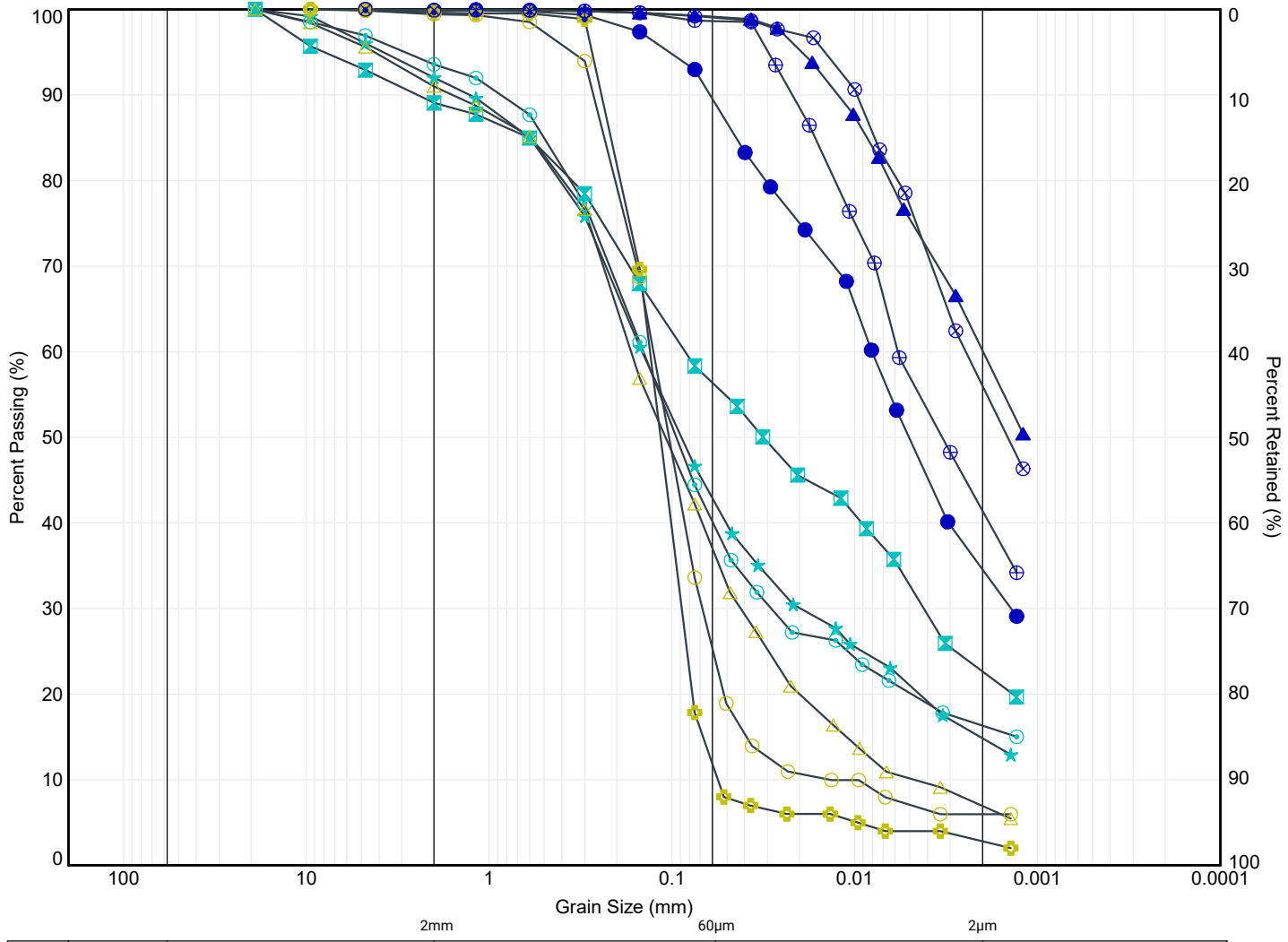
50 mm dia. monitoring well installed.
No. 10 screen

GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Apr 12, 2022	1.9	254.9
May 6, 2022	1.7	255.1

APPENDIX B





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

- 1 Fill
- 4 Clays and Silts
- 2 Upper Till
- 3 Sands and Silts

MIT SYSTEM

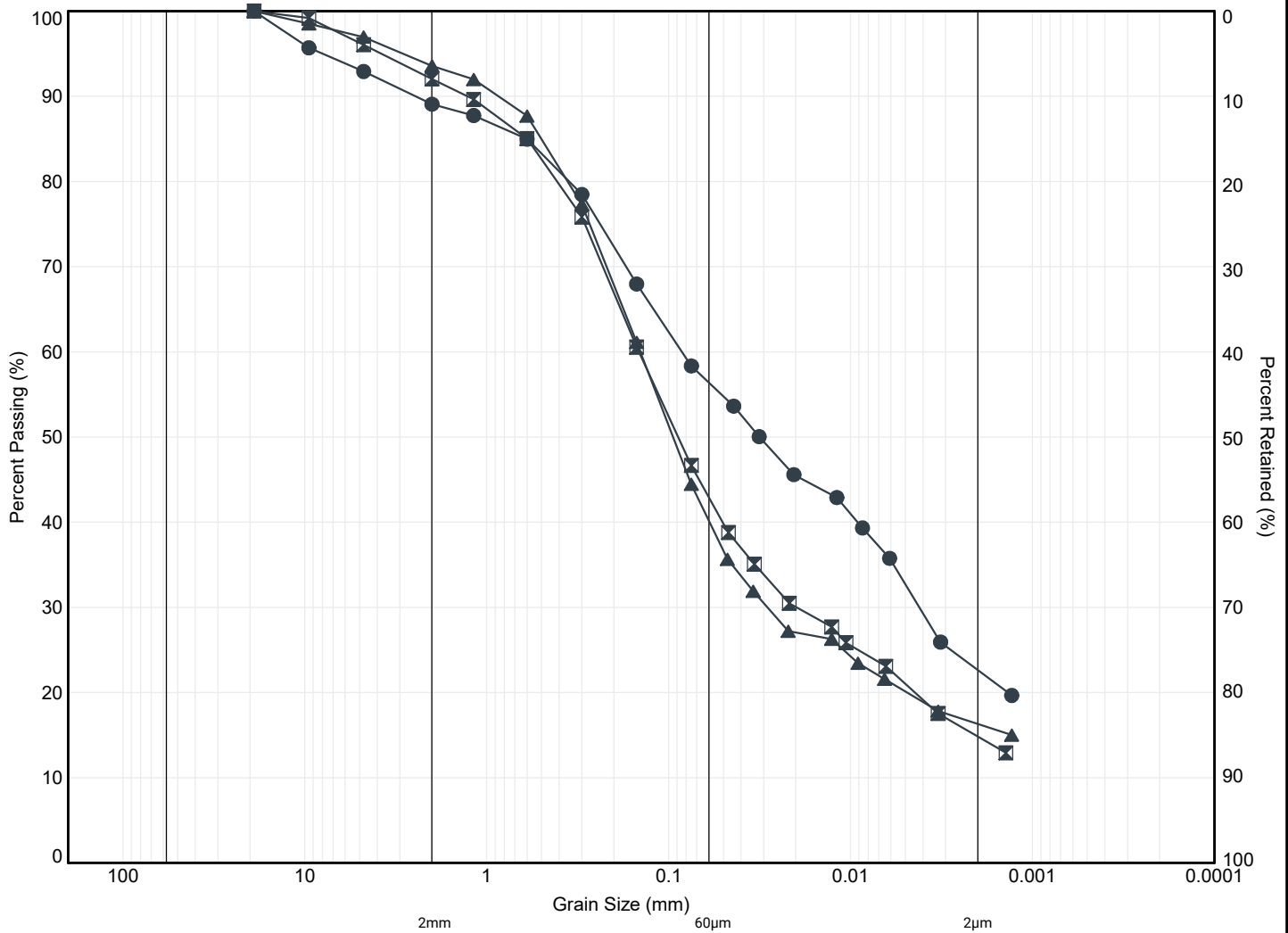
	Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	1	SS10	10.9	245.0	0	10	55	35
■	2	SS6	4.9	249.6	11	32	34	23
▲	2	SS11	12.5	242.0	0	1	39	60
★	4	SS5	3.4	252.5	8	49	28	15
⊙	5	SS4	2.6	252.5	6	54	24	16
⊕	6	SS9	9.4	246.2	0	88	9	3
○	9	SS6	4.9	251.5	1	74	19	6
△	11	SS7	6.3	252.2	9	54	30	7
⊗	11	SS13	15.5	243.0	0	1	43	56
⊕	12	SS10	11.0	245.8	0	2	57	41

file: 22-014.grint.gpi



Title: **GRAIN SIZE DISTRIBUTION
ALL INTERPRETED PROJECT UNITS**

File No.: **22-014**



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

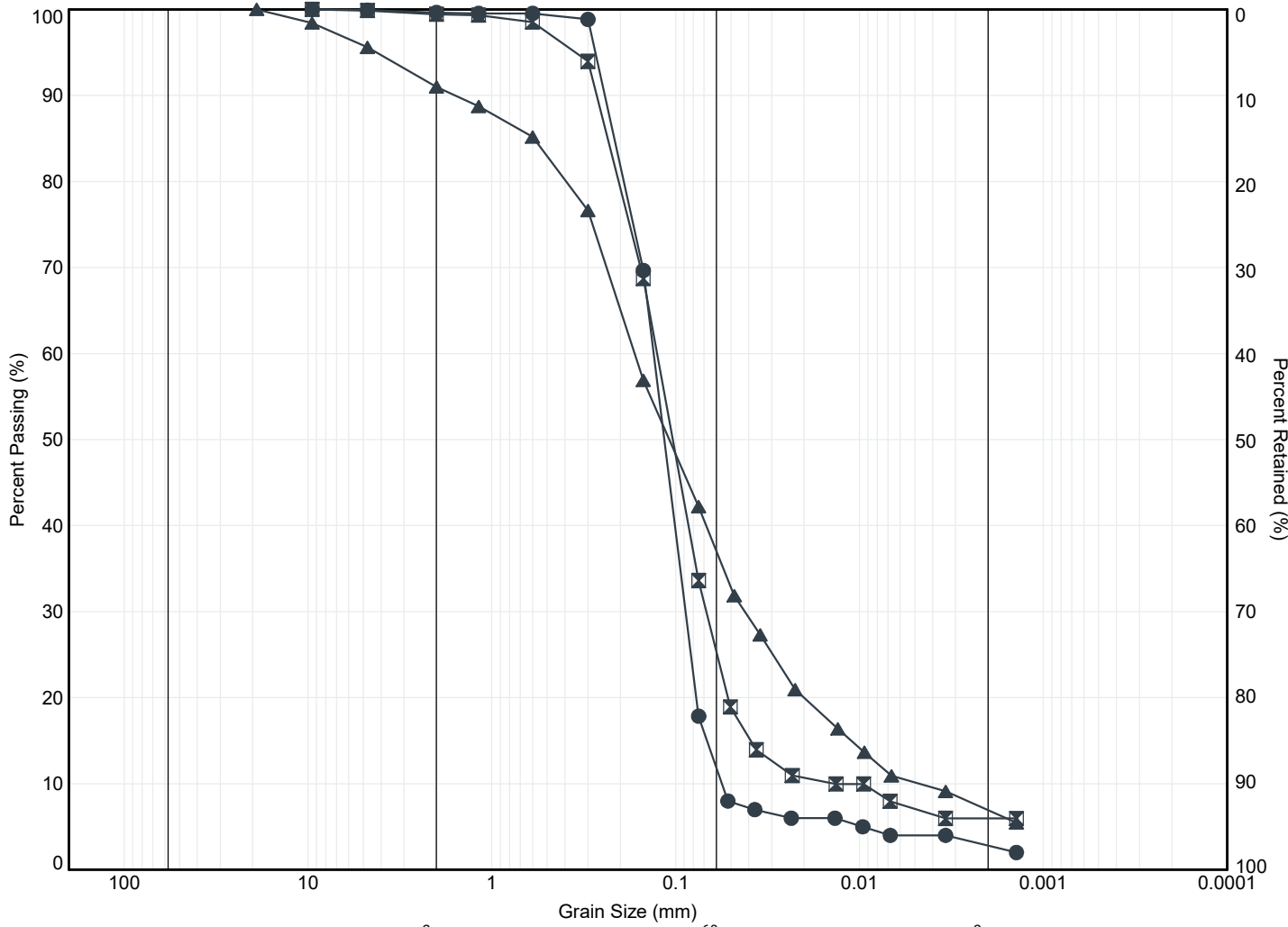
MIT SYSTEM

	Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	2	SS6	4.9	249.6	11	32	34	23
⊠	4	SS5	3.4	252.5	8	49	28	15
▲	5	SS4	2.6	252.5	6	54	24	16

file: 22-014.grit.gpi



Title:	GRAIN SIZE DISTRIBUTION UPPER TILL
File No.:	22-014



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

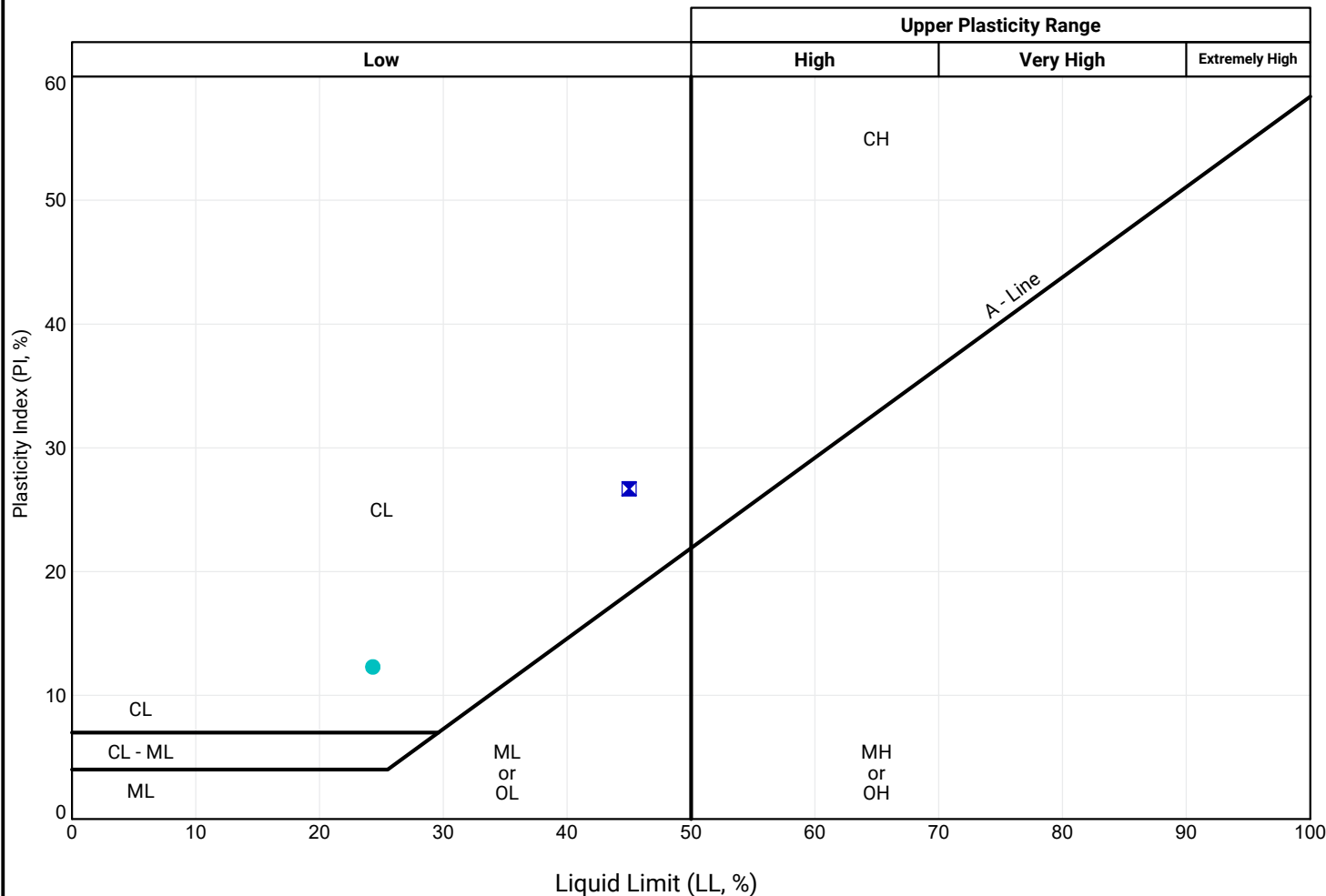
MIT SYSTEM							
Borehole	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
● 6	SS9	9.4	246.2	0	88	9	3
☒ 9	SS6	4.9	251.5	1	74	19	6
▲ 11	SS7	6.3	252.2	9	54	30	7

file: 22-014.grit.gpi



Title: **GRAIN SIZE DISTRIBUTION SANDS AND SILTS**

File No.: **22-014**



- 1 Fill
- 2 Upper Till
- 3 Sands and Silts
- 4 Clays and Silts

Borehole	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)
● 2	SS6	4.9	249.6	24	12	12
⊠ 11	SS13	15.5	243.0	45	18	27

APPENDIX C





Grounded Engineering Inc
ATTN: Deepak Kanraj
1 Banigan Drive
TORONTO ON M4H 1G3

Date Received: 07-APR-22
Report Date: 04-MAY-22 10:05 (MT)
Version: FINAL REV. 2

Client Phone: 647-264-7932

Certificate of Analysis

Lab Work Order #: L2697565
Project P.O. #: NOT SUBMITTED
Job Reference: 22-014
C of C Numbers: 20-955235
Legal Site Desc:

Comments: 04-MAY-22 Revising Sample IDs per client request. -A.Overholster
BH101-SS8 -> BH1-SS8
BH104-SS6 -> BH4-SS6
BH105-SS6 -> BH5-SS6
BH106-SS5 -> BH6-SS5
BH111-SS5 -> BH11-SS5

Amanda Overholster
Account Manager

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

ADDRESS: 5730 Coopers Avenue, Unit #26, Mississauga, ON L4Z 2E9 Canada | Phone: +1 905 507 6910 | Fax: +1 905 507 6927
ALS CANADA LTD Part of the ALS Group An ALS Limited Company

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2697565-1 BH1- SS8 Sampled By: DK on 05-APR-22 @ 14:00 Matrix: SOIL							
Physical Tests							
Conductivity	0.178		0.0040	mS/cm		12-APR-22	R5762035
% Moisture	8.80		0.25	%	07-APR-22	08-APR-22	R5758499
pH	8.04		0.10	pH units		08-APR-22	R5759376
Redox Potential	265		-1000	mV		13-APR-22	R5762455
Resistivity	5610		1.0	ohm*cm		12-APR-22	
Leachable Anions & Nutrients							
Chloride	6.0		5.0	ug/g	08-APR-22	11-APR-22	R5761555
Anions and Nutrients							
Sulphate	81		20	ug/g	08-APR-22	11-APR-22	R5761555
Inorganic Parameters							
Acid Volatile Sulphides	0.37		0.20	mg/kg	08-APR-22	08-APR-22	R5759705
L2697565-2 BH4- SS6 Sampled By: DK on 05-APR-22 @ 14:00 Matrix: SOIL							
Physical Tests							
Conductivity	0.187		0.0040	mS/cm		12-APR-22	R5762035
% Moisture	10.6		0.25	%	07-APR-22	08-APR-22	R5758499
pH	7.97		0.10	pH units		08-APR-22	R5759376
Redox Potential	286		-1000	mV		13-APR-22	R5762455
Resistivity	5350		1.0	ohm*cm		12-APR-22	
Leachable Anions & Nutrients							
Chloride	<5.0		5.0	ug/g	08-APR-22	11-APR-22	R5761555
Anions and Nutrients							
Sulphate	71		20	ug/g	08-APR-22	11-APR-22	R5761555
Inorganic Parameters							
Acid Volatile Sulphides	0.22		0.20	mg/kg	08-APR-22	08-APR-22	R5759705
L2697565-3 BH5- SS6 Sampled By: DK on 05-APR-22 @ 14:00 Matrix: SOIL							
Physical Tests							
Conductivity	0.125		0.0040	mS/cm		12-APR-22	R5762035
% Moisture	10.1		0.25	%	07-APR-22	08-APR-22	R5758499
pH	8.00		0.10	pH units		08-APR-22	R5759376
Redox Potential	272		-1000	mV		13-APR-22	R5762455
Resistivity	8010		1.0	ohm*cm		12-APR-22	
Leachable Anions & Nutrients							
Chloride	20.3		5.0	ug/g	08-APR-22	11-APR-22	R5761555
Anions and Nutrients							
Sulphate	<20		20	ug/g	08-APR-22	11-APR-22	R5761555
Inorganic Parameters							
Acid Volatile Sulphides	0.58		0.20	mg/kg	08-APR-22	08-APR-22	R5759705
L2697565-4 BH6- SS5 Sampled By: DK on 05-APR-22 @ 14:00 Matrix: SOIL							

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

ALS ENVIRONMENTAL ANALYTICAL REPORT

Sample Details/Parameters	Result	Qualifier*	D.L.	Units	Extracted	Analyzed	Batch
L2697565-4 BH6- SS5 Sampled By: DK on 05-APR-22 @ 14:00 Matrix: SOIL							
Physical Tests							
Conductivity	0.283		0.0040	mS/cm		12-APR-22	R5762035
% Moisture	9.62		0.25	%	07-APR-22	08-APR-22	R5758499
pH	8.01		0.10	pH units		08-APR-22	R5759376
Redox Potential	283		-1000	mV		13-APR-22	R5762455
Resistivity	3530		1.0	ohm*cm		12-APR-22	
Leachable Anions & Nutrients							
Chloride	119		5.0	ug/g	08-APR-22	11-APR-22	R5761555
Anions and Nutrients							
Sulphate	29		20	ug/g	08-APR-22	11-APR-22	R5761555
Inorganic Parameters							
Acid Volatile Sulphides	0.32		0.20	mg/kg	08-APR-22	08-APR-22	R5759705
L2697565-5 BH11- SS5 Sampled By: DK on 05-APR-22 @ 14:00 Matrix: SOIL							
Physical Tests							
Conductivity	0.128		0.0040	mS/cm		12-APR-22	R5762035
% Moisture	9.21		0.25	%	07-APR-22	08-APR-22	R5758499
pH	7.95		0.10	pH units		08-APR-22	R5759376
Redox Potential	292		-1000	mV		13-APR-22	R5762455
Resistivity	7800		1.0	ohm*cm		12-APR-22	
Leachable Anions & Nutrients							
Chloride	35.6		5.0	ug/g	08-APR-22	11-APR-22	R5761555
Anions and Nutrients							
Sulphate	<20		20	ug/g	08-APR-22	11-APR-22	R5761555
Inorganic Parameters							
Acid Volatile Sulphides	0.42		0.20	mg/kg	08-APR-22	08-APR-22	R5759705

* 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
<p>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.</p> <p>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).</p>			
EC-WT	Soil	Conductivity (EC)	MOEE E3138
<p>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.</p> <p>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).</p>			
MOISTURE-WT	Soil	% Moisture	CCME PHC in Soil - Tier 1 (mod)
PH-WT	Soil	pH	MOEE E3137A
<p>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.</p> <p>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).</p>			
REDOX-POTENTIAL-WT	Soil	Redox Potential	APHA 2580
<p>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.</p>			
RESISTIVITY-CALC-WT	Soil	Resistivity Calculation	APHA 2510 B
<p>"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.</p>			
SO4-WT	Soil	Sulphate	EPA 300.0
<p>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.</p>			
SULPHIDE-WT	Soil	Sulphide, Acid Volatile	APHA 4500S2J
<p>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.</p>			

** 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:

20-955235

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: L2697565

Report Date: 04-MAY-22

Page 1 of 3

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

Contact: Deepak Kanraj

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
CL-R511-WT	Soil							
Batch	R5761555							
WG3715270-3	CRM	AN-CRM-WT						
Chloride			99.5		%		70-130	11-APR-22
WG3715270-4	DUP	WG3715270-5						
Chloride		5.5	<5.0	RPD-NA	ug/g	N/A	30	11-APR-22
WG3715270-2	LCS							
Chloride			99.9		%		80-120	11-APR-22
WG3715270-1	MB							
Chloride			<5.0		ug/g		5	11-APR-22
EC-WT	Soil							
Batch	R5762035							
WG3715354-8	DUP	L2697339-3						
Conductivity		0.126	0.128		mS/cm	1.9	20	12-APR-22
WG3715354-7	IRM	WT SAR4						
Conductivity			102.8		%		70-130	12-APR-22
WG3716493-1	LCS							
Conductivity			95.2		%		90-110	12-APR-22
WG3715354-6	MB							
Conductivity			<0.0040		mS/cm		0.004	12-APR-22
MOISTURE-WT	Soil							
Batch	R5758499							
WG3714864-3	DUP	L2696983-10						
% Moisture		16.0	17.3		%	7.9	20	08-APR-22
WG3714864-2	LCS							
% Moisture			99.2		%		90-110	08-APR-22
WG3714864-1	MB							
% Moisture			<0.25		%		0.25	08-APR-22
PH-WT	Soil							
Batch	R5759376							
WG3714951-1	DUP	L2697413-2						
pH		7.25	7.25	J	pH units	0.00	0.3	08-APR-22
WG3715077-1	LCS							
pH			7.01		pH units		6.9-7.1	08-APR-22
REDOX-POTENTIAL-WT	Soil							
Batch	R5762455							
WG3716982-1	CRM	WT-REDOX						
Redox Potential			103.4		%		90-110	13-APR-22
WG3715051-1	DUP	L2697339-1						



Quality Control Report

Workorder: L2697565

Report Date: 04-MAY-22

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

Contact: Deepak Kanraj

Test	Matrix	Reference	Result	Qualifier	Units	RPD	Limit	Analyzed
REDOX-POTENTIAL-WT	Soil							
Batch	R5762455							
WG3715051-1	DUP	L2697339-1						
Redox Potential		291	280		mV	3.9	25	13-APR-22
SO4-WT	Soil							
Batch	R5761555							
WG3715270-3	CRM	AN-CRM-WT						
Sulphate			95.7		%		60-140	11-APR-22
WG3715270-4	DUP	WG3715270-5						
Sulphate		47	46		ug/g	2.3	25	11-APR-22
WG3715270-2	LCS							
Sulphate			100.0		%		70-130	11-APR-22
WG3715270-1	MB							
Sulphate			<20		ug/g		20	11-APR-22
SULPHIDE-WT	Soil							
Batch	R5759705							
WG3715346-3	DUP	L2697574-2						
Acid Volatile Sulphides		0.26	0.27		mg/kg	4.2	45	08-APR-22
WG3715346-2	LCS							
Acid Volatile Sulphides			80.8		%		70-130	08-APR-22
WG3715346-1	MB							
Acid Volatile Sulphides			<0.20		mg/kg		0.2	08-APR-22

Quality Control Report

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Contact: Deepak Kanraj

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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.

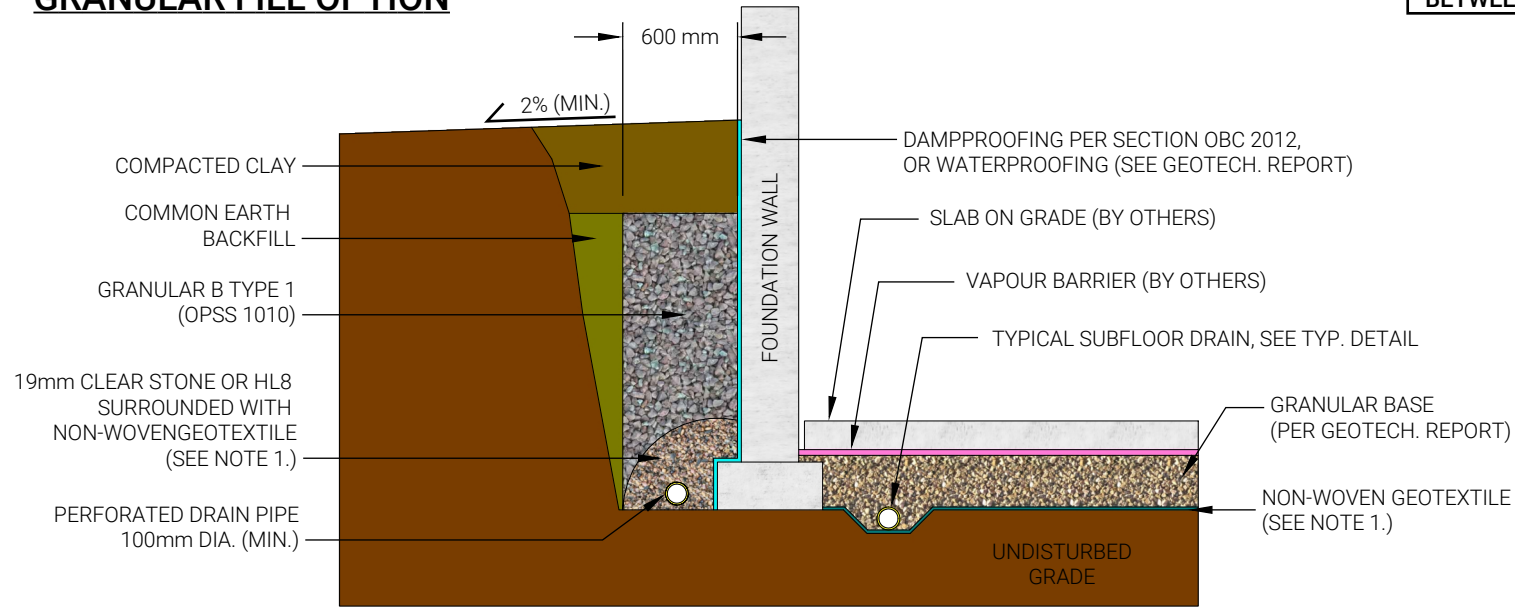
The ALS Quality Control Report is provided to ALS clients upon request. ALS includes comprehensive QC checks with every analysis to ensure our high standards of quality are met. Each QC result has a known or expected target value, which is compared against pre-determined data quality objectives to provide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

APPENDIX D

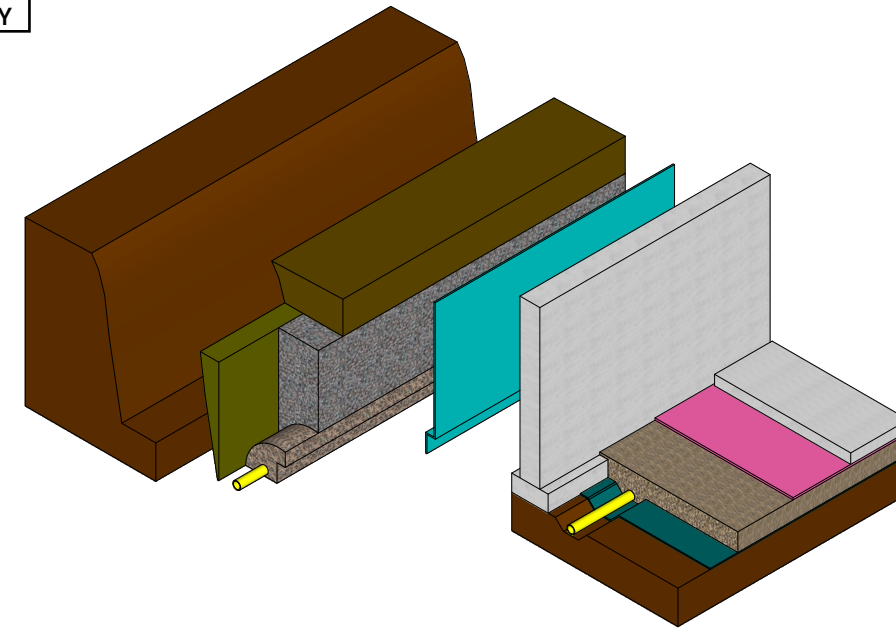


GRANULAR FILL OPTION



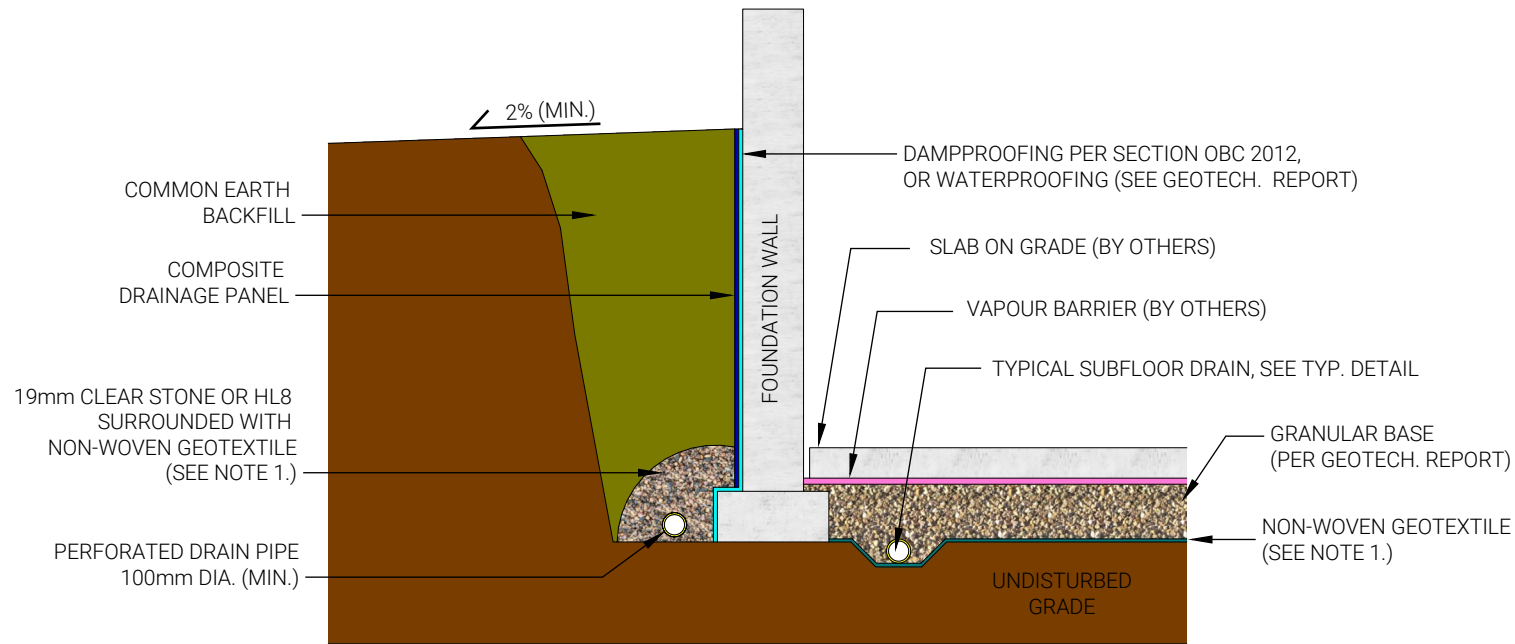
SECTIONAL VIEW

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY

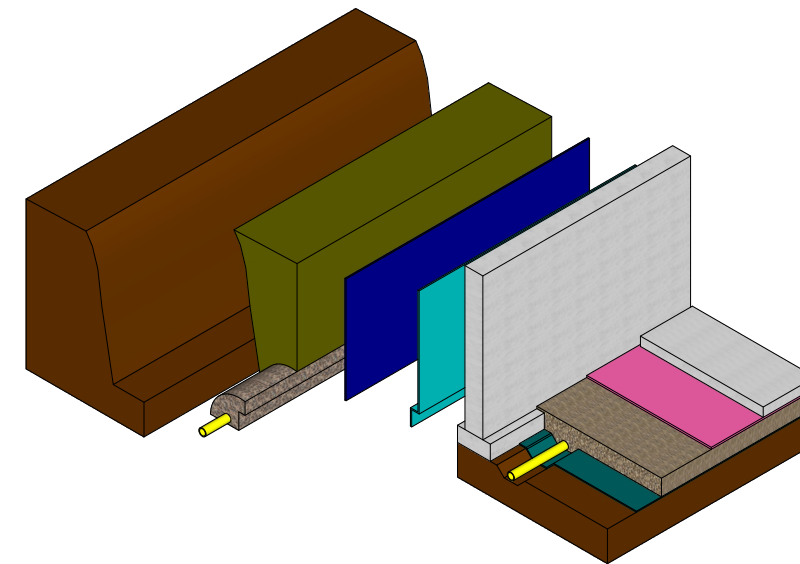


ISOMETRIC VIEW

GEO-COMPOSITE DRAINAGE PANEL OPTION



SECTIONAL VIEW



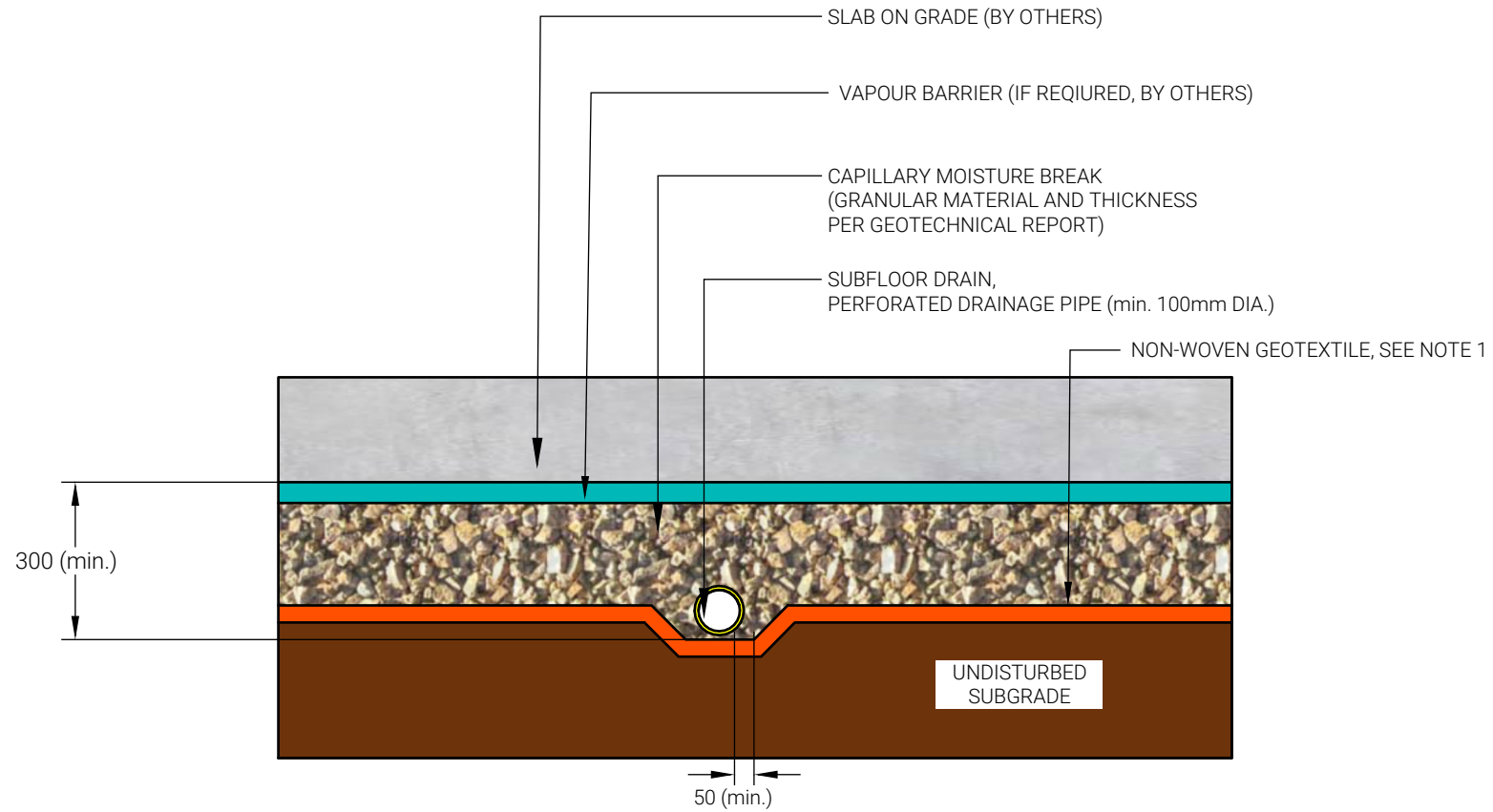
ISOMETRIC VIEW

NOTES

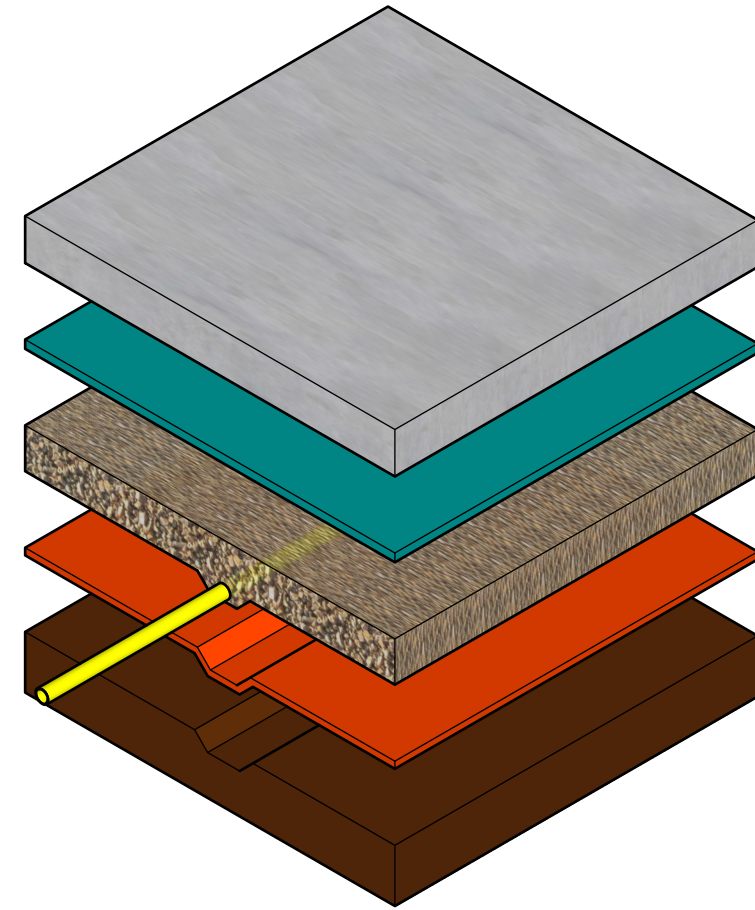
1. A NON-WOVEN GEOTEXTILE WITH AN APPARENT OPENING SIZE OF < 0.250mm AND A TEAR RESISTANCE OF > 200 N.

Title

OBJECTS ARE COLOR-CODED
BETWEEN TWO VIEWS FOR CLARITY



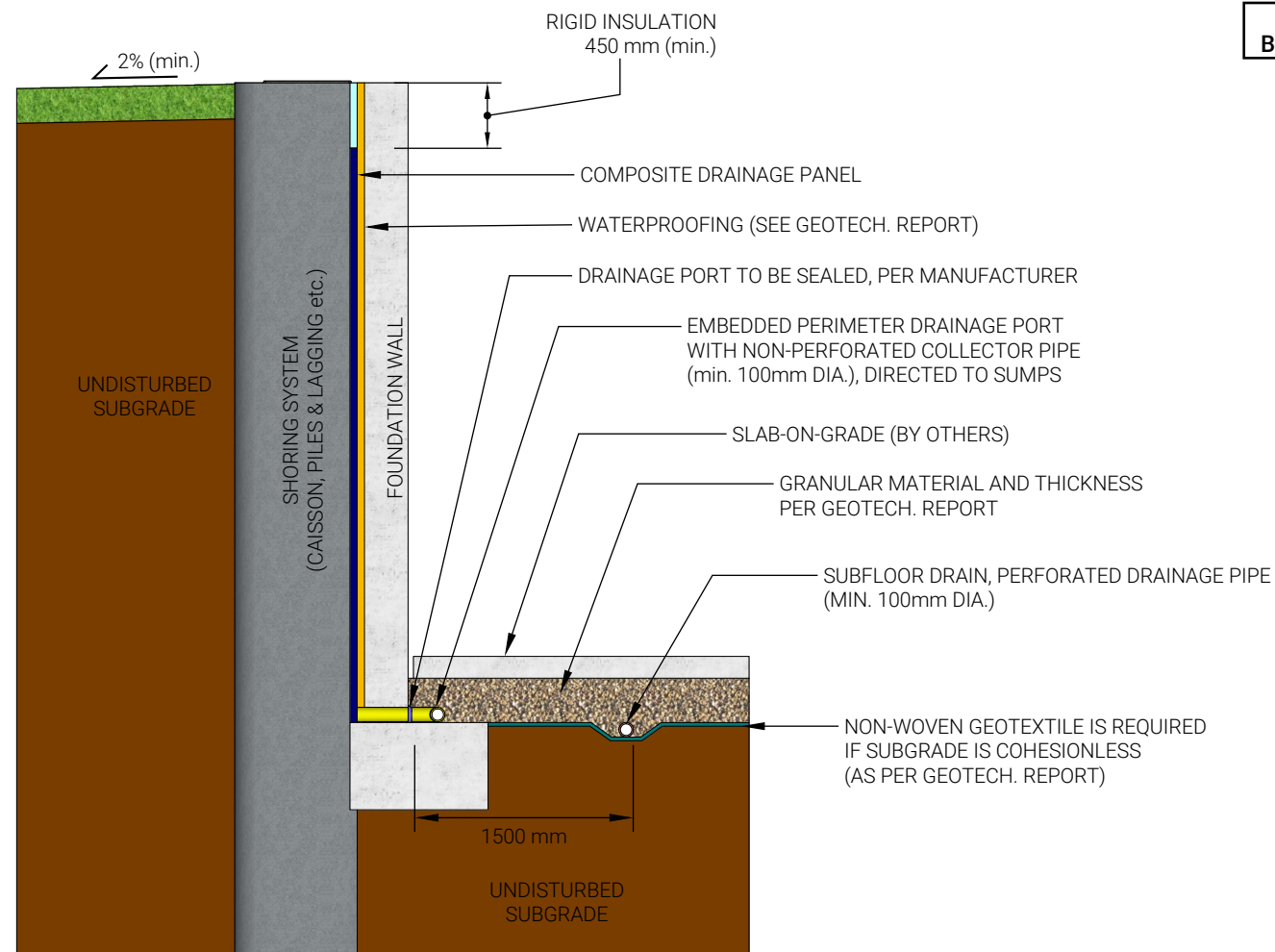
SECTIONAL VIEW



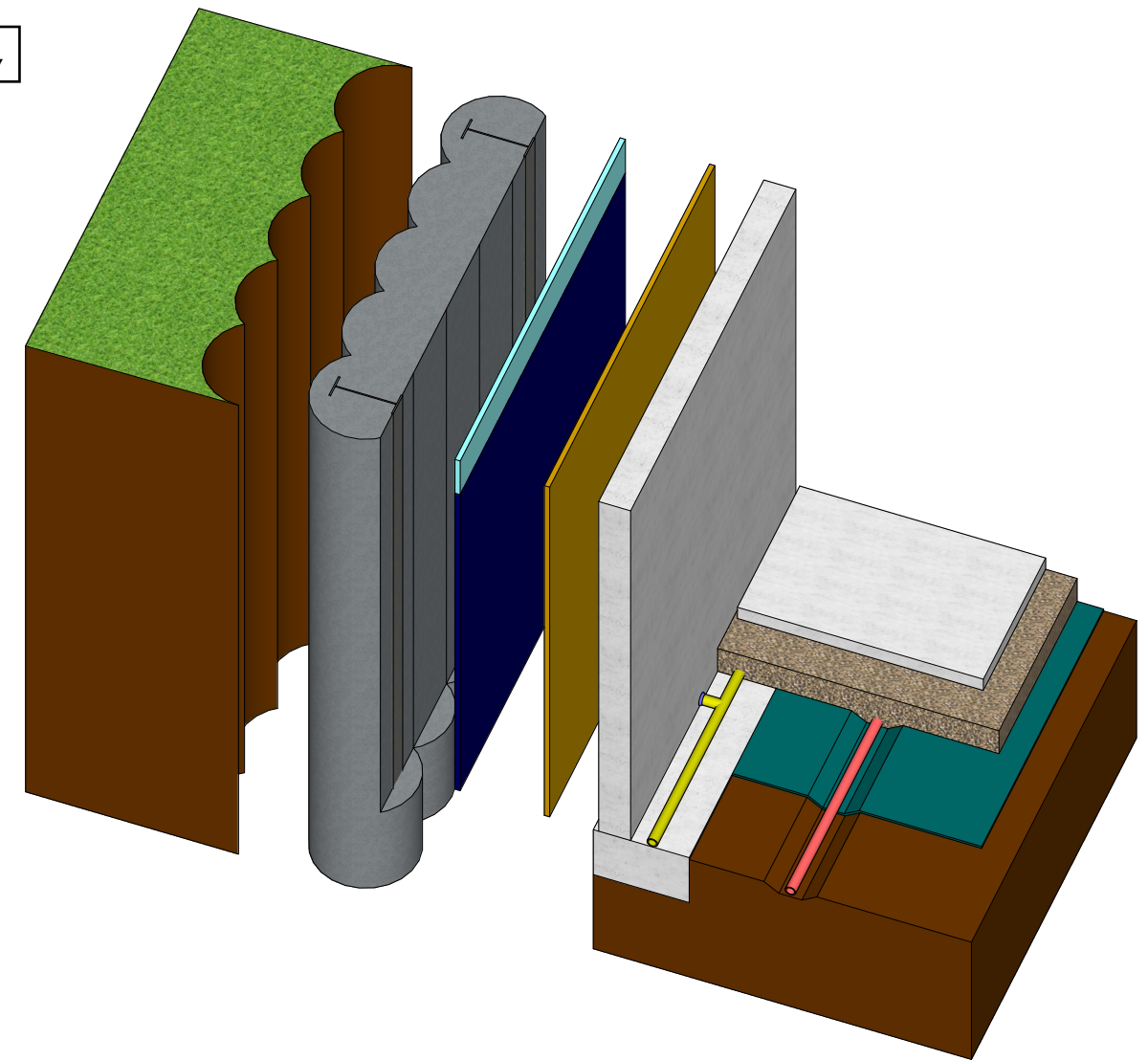
ISOMETRIC VIEW

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 $< 0.250\text{mm}$ AND A TEAR RESISTANCE OF $> 200\text{ N}$).
2. TYPICAL SCHEMATIC ONLY. MUST BE READ IN CONJUNCTION WITH GEOTECHNICAL REPORT.



OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW

ISOMETRIC 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.
4. A NON-WOVEN GEOTEXTILE MUST SEPARATE THE SUBGRADE FROM THE SUBFLOOR DRAINAGE LAYER IF THE SUBGRADE IS COHESIONLESS. THE NON-WOVEN GEOTEXTILE MAY CONSIST OF TERRAFIX 360R OR AN APPROVED EQUIVALENT.

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.

Title



BASEMENT DRAINAGE SHORING SYSTEM TYPICAL DETAILS