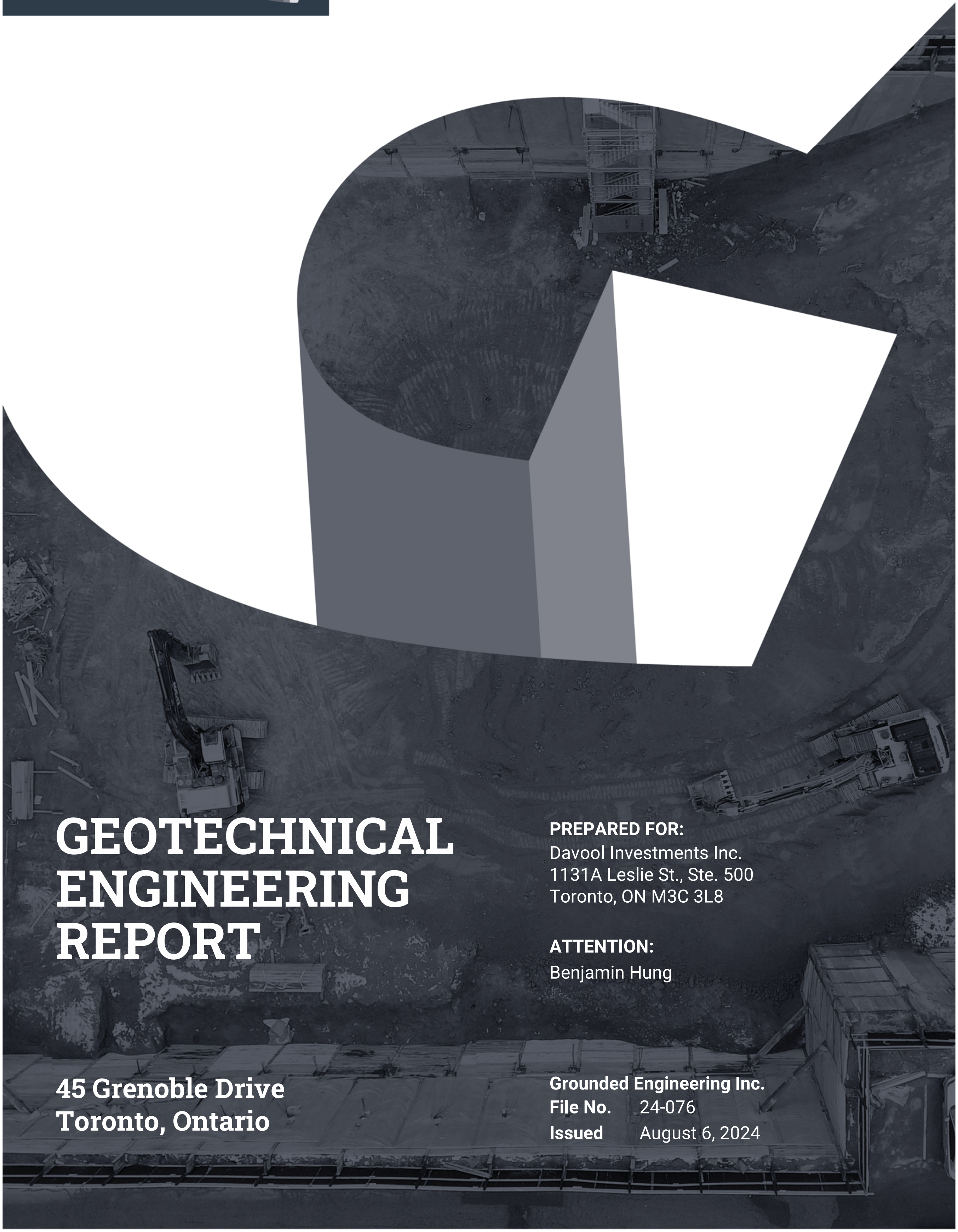

GEOTECHNICAL ENGINEERING REPORT

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Grounded Engineering Inc.
File No. 24-076
Issued August 6, 2024

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1 Introduction

Davool Investments Inc.has retained Grounded Engineering Inc. to provide geotechnical engineering design advice, in accordance with the City of Toronto Terms of Reference for Geotechnical Study, for their proposed development at 45 Grenoble Drive, in Toronto, Ontario. The level of study presented in this report is consistent with the requirements for a Zoning Bylaw Amendment, Plan of Subdivision, Consent to Server, or Site Plan Control application. Deep drilling and pressuremeter testing is excluded from the current scope of work. Additional boreholes, in-situ testing, and a detailed geotechnical engineering report will be required for detailed foundation design and building permit purposes.

There is an existing 28-storey building with two levels of underground parking across the site, and under the proposed basement footprint. The existing tower will remain.

The proposed project includes the construction of a new 39± storey infill tower, with a P3 underground parking structure beneath the new tower footprint. The proposed P3 FFE is set at 119.21 m. The existing underground structure will therefore be lowered from a P2 to a P3 in that location.

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

- Site survey, prepared by JD Barnes (Mar 20, 2023).
- Architectural Drawings, “45 Grenoble Drive, Toronto, Ontario”; Project 23009, dated May 22, 2024 (Issued for rezoning application), prepared by BDP Quadrangle Limited.

Grounded’s subsurface investigation of the site to date includes four (4) boreholes (Boreholes 101 to 104) with seven (7) monitoring wells, which were advanced from May 27th to 29th, 2024.

Based on the borehole findings, preliminary geotechnical engineering advice for the proposed development is provided for foundations, seismic site classification, earth pressure design, slab on grade design, and basement drainage. 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 (as established on existing site survey). The horizontal coordinates are provided relative to the Universal Transverse Mercator (UTM) geographic coordinate system.

For Boreholes 103 and 104, SPT N values were obtained using a 32 kg hammer falling 760 mm. The results on the logs have been corrected to equivalent N values based on a 63.5 kg hammer falling 760 mm based on ASTMD1586.

2.1 Stratigraphy

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

2.1.1 Surficial and Earth Fill

Surficial fill (pavements, aggregate, topsoil, etc.) thicknesses were observed in individual borehole locations through the top of the open borehole. Thicknesses may vary between and beyond each borehole location.

The exterior boreholes (Borehole 101, 102) encountered 40 to 50 mm of topsoil at ground surface. Boreholes 103 and 104 were drilled inside the parking garage and encountered a 90 mm thick concrete slab structure. No aggregate base course was observed under the existing slab on grade in the borehole locations.

Underlying the surficial materials, the exterior boreholes observed a layer of earth fill that extends to Elev. 125.5 to 125.1 metres. The earth fill varies in composition but generally consists of sand with some silt and trace gravel. It contains trace rootlets. The earth fill is typically brown, and moist. The interior drilling locations did not encounter earth fill materials.

Standard Penetration Test (SPT) results (N-Values) measured in the fill range from 5 to 30 blows per 300 mm of penetration (“bpf”). Due to inconsistent placement and the inherent heterogeneity of earth fill materials, the relative density of the earth fill could be variable.

2.1.2 Upper Sands

Underlying the fill materials or the existing slab on grade, all boreholes encountered an undisturbed deposit of native sand (the “**upper sands**” unit). The top of this unit was encountered at Elev. 125.5 to 125.1 m, or directly below the existing slab at the interior locations. It extends down to Elev. 120.5 to 119.2 m. The upper sands are brown and moist to wet. This unit contains some gravel, trace silt and trace clay. SPT N-values measured in the upper sands unit range from 15 to 40 bpf (compact to dense).

2.1.3 Glacial Till

Underlying the upper sands in all boreholes, a glacial till was encountered with a matrix of cohesive clayey silt (the “**glacial till**” unit). This unit was encountered at elevations of 120.5 to 119.2 m and extends down to elevations 116.0 to 111.0 m. Borehole 103 investigation depth did not extend below this layer. The glacial tills are grey and moist. This unit contains some sand and trace gravel. SPT N-values measured in this unit range from 10 to over 50 bpf (stiff to hard).

2.1.4 Silts and Clays

Underlying the glacial till in Boreholes 101, 102, and 104 a deposit of clay and silt was encountered. These soils are grouped together as the **“silts and clays”** unit. It was encountered at elevations of 116.0 to 111.0 m and extends down below investigation depth in Boreholes 101, 102, and 104. The silts and clays are generally grey and moist containing some sand and trace gravel. SPT N-values measured in this unit range from 28 to over 50 bpf (very stiff to hard).

2.1.5 Estimated Bedrock Elevation

The elevation of bedrock was not encountered in the boreholes on this site. In this general area of Toronto, bedrock has been encountered at approximate Elev. 85± m based on publicly available data as well as nearby sites across the street investigated by Grounded Engineering (now publicly available) which provides preliminary information for bedrock elevation/depth. Future boreholes are required at detailed design to determine bedrock elevation and quality at this site.

The bedrock in this area of Toronto is the Georgian Bay Formation, which comprises thin to medium bedded grey shale and limestone of Ordovician age. The fissile shale is interbedded with non-fissile calcareous shale, limestone, dolostone, and calcareous sandstone (conventionally grouped together as “limestone”) which are typically laterally discontinuous. Per the appended terminology, the Georgian Bay shale is typically classified as “weak” whereas the limestone interbedding is classified as “medium strong to strong”.

2.2 Groundwater

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

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

Well ID	Well Diameter (mm)	Ground Surface (masl)	Top of Screen (masl)	Bottom of Screen (masl)	Screened Geological Unit
BH101-S	50	128.1	125.0	122	Sand
BH101-D	50	128.1	115.9	112.9	Silts and Clays
BH102-S	50	127.8	124.8	121.7	Sand
BH102-I	50	127.8	120.2	117.1	Silts and Clays
BH102-D	50	127.8	112.6	109.5	Silts and Clays
BH103	50	122.2	116.1	113.0	Silts and Clays
BH104	50	122.1	116.0	113.0	Silts and Clays

Borehole No.	Borehole depth (m)	Upon completion of drilling		Water Level in Well, highest (m)	
		Depth to cave (m)	Unstabilized water level (m)	Date	Depth/Elev.
BH101-S	15.7	n/a	Dry	2024/6/6	Dry
BH101-D				2024/7/5	7.0/121.1
BH102-S	18.7	n/a	Filled with drill water	2024/7/5	Dry
BH102-I				2024/7/19	**6.5/121.4**
BH102-D				2024/7/5	15.2/112.6
BH103	9.4	n/a	Filled with drill water	2024/7/5	5.5/116.7
BH104	9.4	n/a	Filled with drill water	2024/7/5	4.2/117.9

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. 121.4 m. The City of Toronto Maximum Anticipated Groundwater Level (MAGWL) is a planning elevation to determine whether or not the City will require a watertight below-grade structure, and is provided in the hydrogeological report.

The cohesionless upper sands unit (high permeability soils) is within the zone of excavation and will produce free-flowing water when penetrated. The silts and clays and glacial till have a low permeability and will yield minor seepage only in the long-term.

Grounded has prepared a hydrogeological report for this site (File No. 24-076).

3 Preliminary Geotechnical Engineering Recommendations

Based on the factual data summarized above, preliminary geotechnical engineering recommendations are provided. These preliminary recommendations are for due diligence and planning application purposes only. They must be supplemented and confirmed by additional boreholes, wells, and a detailed geotechnical engineering report at the detailed design stage.

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.

Per Toronto Water's Infrastructure Management's Policy on Managing Foundation Drainage (November 1, 2021), long-term discharge of foundation drainage to the City's sewer system will not be permitted unless there is an exemption.

As part of their policy, the City has defined a Maximum Anticipated Ground Water Level (MAGWL), which is the highest measured groundwater table elevation measured plus a regulatory offset called the "fluctuation allowance". The fluctuation allowance is based in part on the month in which the highest groundwater level measurement was made. The MAGWL is not a design groundwater table for engineering purposes, it is merely a planning elevation that the City uses to assess whether it will require a watertight below-grade structure or not.

The relevant groundwater information is summarized as follows:

- Design groundwater table for engineering purposes: Elev. 121.4 m
- Lowest Proposed FFE: Elev. 119.21± m
- The design groundwater table is above the lowest FFE.
- Therefore, a watertight below-grade structure may be anticipated for new basement structures.
- If the existing foundation walls are to be maintained, an **exemption** will be required to allow the new basement to be made fully drained.

3.1 Foundation Design Parameters

The proposed development will consist of one 39-storey infill tower, with a P3 underground parking structure set below the tower footprint at an estimated lowest FFE of 119.21± m. The following foundation options have been considered in our analysis.

- Conventional spread footings - Podium structures only.
- Raft foundation(s) – Towers and/or podium structures. Preliminary advice provided.
- Caissons to rock (elevation of rock not determined) - Preliminary advice provided.

Footings stepped from one elevation to another should be offset at a slope not steeper than 7 vertical to 10 horizontal. This requirement exists to avoid undermining adjacent footings at the higher elevation.

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

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

3.1.1 Spread Footings

Foundations made for the proposed P3 level about 1± m below the P3 Elev. (119.21 m) will bear on undisturbed very stiff to hard glacial till. Conventional spread footings made to bear on this soil may be designed using a maximum factored geotechnical resistance at ULS of 450 kPa. The geotechnical reaction at SLS is 300 kPa, for an estimated total settlement of 25 mm.

The capacities provided above is based on an individual spread footing foundations that are 1 m wide and embedded a minimum of 1 m 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 an estimated settlement which for practical purposes is linear and non-recoverable. Differential settlement is related to column spacing, column loads, and footing sizes.

Higher capacity spread footings are available if larger and deeper footings are designed. These can be provided on request. We have targeted spread footings for podium support only, as maximal spread footing capacities will still not be adequate for the support a 39-storey tower.

3.1.2 Raft Foundation

The following advice is preliminary only. If a raft foundation approach is the preferred foundation option, additional deep boreholes with pressuremeter testing to measure the in situ soil stiffness are required.

The available spread footing capacities will not be sufficient to support the proposed tower. A raft foundation may be considered. A raft foundation can be used in conjunction with a watertight basement strategy, including watertight foundation walls designed to withstand hydrostatic forces (lateral and uplift). Alternatively, it can be used in conjunction with a drained basement approach, for structural support only.

A 33 x 33 m triangular raft underlying the tower is considered in the bearing capacity discussion below. Raft slabs for a podium structure will be subjected to much less load, and will not govern design.

Considering a lowest P3 FFE of 119.21± m, it is assumed that a raft would be founded around 2 m lower (Elev. 117.21± m), on undisturbed very stiff to hard native cohesive tills.

The preliminary raft design parameters assume a uniform load at the base of the raft. In reality, raft loads are non-uniform; they are typically highest at the core and lowest at the perimeter. The preliminary parameters below are provided as the initial step in determining raft feasibility (a structural task). The detailed design process is described below.

Bulk excavation to underside of raft elevation (Elev. 117.21 m) will induce a reduction in effective stress of 180 kPa, which is the unload stress. Utilizing preliminary soil stiffness parameters, analysis of a uniformly loaded raft foundation shows that a uniform total applied SLS bearing pressure of 260 kPa (incorporating a 0.9 factor as per the CFEM 5th edition) at the base of the raft will generate an estimated 25± mm of settlement. Similarly, a uniform geotechnical reaction at SLS of 400 kPa will generate an estimated 50± mm of settlement.

The modulus of subgrade reaction for design of a raft slab is a function of the size of the raft, the applied load, and whether loading is within the recompression range or the virgin range. On the basis of our preliminary stiffness parameters and the assumption of uniform raft loading, the preliminary modulus of subgrade reaction appropriate for 33x33 m raft design at this site is about 5,900 kPa/m for loads over 180 kPa SLS.

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

The maximum factored geotechnical resistance at ULS of this 33 x 33 m raft foundation is 800 kPa. Raft foundation design is typically governed by service load criteria.

Detailed raft design is an iterative process between the structural and the geotechnical engineer. 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 of SLS pressures). Grounded then models that non-uniform pressure distribution to more accurately estimate the settlement at each point under the raft. The resulting estimated settlement distribution is then sent back to the structural engineer to assess the total and differential settlements under the raft, as well as lateral impacts on adjacent footings and structures. The structural design is then modified as required.

If the raft slab is to be fully watertight, the structure must be designed to resist uplift and lateral hydrostatic pressure on foundation walls. During construction, it will be necessary to consider the potential uplift pressure on the underside of a raft foundation due to hydrostatic forces. Dewatering operations during construction 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 121.4 m is to be used.

Differential settlement is related to real non-uniform raft load distribution and must be assessed as part of the detailed design process. Impacts to adjacent structures caused by settlement within the raft's zone of influence will also need to be reviewed.

Tiedown Anchors for Rafts

If deemed necessary by the structural engineer, micropile tiedowns can be designed to resist uplift. In the very dense subgrade below founding elevation, post-grouted micropile anchors in tension can be designed using a maximum factored geotechnical resistance at ULS of 45 kN/m of adhered anchor length (at a nominal diameter of 150 mm).

One or more prototype anchors must be performance-tested to demonstrate the anchor capacity and validate design assumptions for these permanent tiedowns, per OPSS 942.07.12.05.02.

The capacity above is provided using a resistance factor of 0.3 for tension without a load test. Provided that a site specific tension load test is performed, the resistance factor will be increased to 0.4, and the micropile capacity can be re-evaluated. After installation, each of the permanent anchors is proof tested to not less than 150% of SLS design load, per OPSS 942.07.12.05.02.

Micropile anchors are made with high-strength hot-rolled threadbar conforming to ASTM A615 or CSA G30.18. For permanent installations they should be made within grouted HDPE corrugated sheaths to provide “double corrosion protection”. Industry-standard grout cover may be used as a corrosion protection mechanism, subject to a review of the corrosivity and sulphate attack data.

Helical pile anchors are also feasible, subject to consultation from the design-build contractor. 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. Adequate corrosion protection must be provided.

In addition to designing the anchors for grout-soil adhesion capacity, global stability must also be checked. Tie-down anchors must also be designed to a depth sufficient to engage the necessary bulk unit weight of soil. Soil anchors are made to engage a 30-45 degree cone of soil per anchor, measured from vertical[1]. The anchor spacing and overlapping zones of influence must be considered. A typical detail is appended.

3.1.3 Caissons to Rock

The following advice is preliminary only since the elevation and quality of bedrock at this site was not investigated at this site, as per the agreed scope of work. If a caisson approach is preferred, rock coring in additional deep boreholes for detailed foundation design is required to observe the elevation of the top of sound bedrock at this site.

End-bearing caissons may also be used to support the proposed structure. If the basement is to be watertight, a caisson approach would require the use of a pressure slab to create a watertight basement.

End-bearing caissons made to bear on unweathered (sound) bedrock may be designed using a maximum factored geotechnical resistance at ULS of 12 MPa. The geotechnical reaction at SLS is 10 MPa, for 20 mm of estimated settlement at pile tip elevation, for individual caissons no larger than 2 m diameter and not subject to group effects.

In addition to the displacement of the rock, there will be compression of the concrete caisson shaft under loading which will increase the apparent settlement at the structure level. Caisson shaft compression must be assessed by the structural engineer.

Caissons should be separated from each other by at least 2 times the largest caisson diameter (measured on centres) to avoid inducing additional settlement from group effect. Caissons 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. If this situation is unavoidable from a structural engineering perspective, we can review the structural drawings and estimate the expected settlement of the caisson group, on request.

There are zones of soil at this site that are sufficiently cohesionless, permeable and wet that augered boreholes for caissons will need to protected against loss of ground, upheave, and basal disturbance due to the ingress of groundwater from the lower pressurized aquifer. Augered boreholes for caissons may require temporary liners, polymer mud drilling techniques, tremie pour concrete, pre-advancing casing, or other means and methods as deemed necessary by the contractor to prevent groundwater inflow or loss of soil into the drill holes, disturbance to placed concrete, or similar issues. Concrete for caissons must be placed by tremie method where there is more than 300 mm of water or fluid at the base of the hole.

The following construction methodology must be utilized for all structural caisson installations:

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[1] FHWA. “Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems.” Publication No. FHWA-IF-99-015, June 1999, Figure 54.

- All caisson excavations are to be inspected on a full-time basis by Grounded per the Ontario Building Code (2012).
- Caissons designed to bear on sound rock are to be initially advanced to the top of weathered bedrock (see Section 2.1.4) but must be confirmed with additional boreholes for detailed foundation design.
- Once the top of weathered bedrock elevation is established for a given caisson by Grounded, the caisson must then be advanced a minimum of 1 m into sound bedrock (elevation to be identified in future boreholes).
- Auger, cleanout bucket, or one-eyed bucket cleaning of the hole base is to then take place in each caisson hole, and visually inspected by Grounded to ensure that base cleaning has been carried out as thoroughly as practically possible.
- Place 30 MPa (min.) concrete to a minimum depth of 600 mm in the base of the hole (volume to be determined based on caisson diameter) to be stirred with the auger without advancing the auger any further for about 5 minutes.
- The auger spun concrete is then removed and wasted, leaving no more than 100 mm depth of concrete at the base of the caisson.
- Tremie placement of concrete is required wherever the drill holes have more than 150 mm of water in the base or are full of drilling fluid.
- Complete construction of the caisson to cut off elevation.

Any recommendations must also satisfy the structural engineering requirements regardless of any interpretation provided herein.

Grounded recommends sonic caliper testing (or equivalent) to confirm verticality and diameter. Grounded generally recommends carrying such tests on the first five (5) caissons, and 10% of the caissons thereafter. The structural engineer should specify the number of tests to verify the quality of the contractor’s installation. Grounded recommends that this testing be carried out on every caisson at this site, prior to the placement of concrete. To confirm concrete placement, thermal integrity profiling (TIP), crosshole logging, or another similar test is recommended. Grounded reserves the right to increase the testing frequency, subject to the results of the initial testing.

3.2 Seismic Site Classification

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 30 metres of the site stratigraphy below spread footing/grade beam elevation, where shear wave velocity (v_s) measurements have been taken. Alternatively, the classification is estimated from the rational analysis of undrained shear strength (s_u) or penetration resistance (N-values) according to the OBC and National Building Code of Canada.

Below the nominal founding elevations (for spread footings or grade beams) of 120± metres, the boreholes observe very stiff to hard soils. 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.

Consideration should be given to conducting a site-specific Multichannel Analysis of Surface Waves (MASW) as part of a future scope of work, to determine the average shear wave velocity in the 30 meters of soil and rock stratigraphy (Vs_{30}) below the proposed FFE. The current OBC anchors the seismic hazard data to the site class provided above; however, the National Building Code 2020 (and the upcoming revision to the OBC) provides the option of calculating the seismic hazard (i.e. spectral acceleration) directly from average Vs_{30} measurement.

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 Sands	20	35	0.27	0.43	3.69
Glacial Till	22	30	0.33	0.50	3.00
Clays and Silts	22	30	0.33	0.50	3.00

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
h = the depth at which P is calculated (m)
K = earth pressure coefficient
γ = soil bulk unit weight (kN/m^3)
γ' = submerged soil unit weight (γ - 9.8 kN/m^3)
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:

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

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

The City of Toronto may require this basement to be fully waterproofed, according to their policy. In this case, the full height of the basement walls should be watertight and designed to withstand horizontal hydrostatic pressure below Elev. 121.4 m.

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

Foundation resistance to sliding is proportional to the friction between the 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 CFEM 5th Ed. (0.8 for cohesionless soils or rock; 0.6 for cohesive soils)
N = normal load at base of footing (kN)
φ = internal friction angle (see table above)

3.4 Slab on Grade Design Parameters

3.4.1 Watertight Option

If the structure is to be fully watertight and designed to withstand uplift and hydrostatic pressures, with no permanent drainage, a conventional slab on grade and drained basement approach will not be adopted at this site and conventional slab-on-grade design parameters do not apply. Design parameters for a raft foundation are provided in Section 3.1. If caissons are to be used, the lowest floor will be made as a pressure slab spanning between foundation elements, to be designed by the structural engineer.

3.4.2 Drained Option

For a drained basement (see Section 3), at the proposed lowest P3 elevation, the undisturbed 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 below). In this case, the slab on grade must be provided with a drainage layer and capillary moisture break, which is achieved by forming the slab on a minimum 300 mm thick layer of 19 mm clear stone (OPSS.MUNI 1004) vibrated to a dense state.

Given the nature of the soils at this site, recompaction or proof rolling of the undisturbed native subgrade will weaken these materials. These activities should be specifically prohibited when preparing native 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 excessive 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

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 requirement for a permanent basement drainage system depends on whether a fully watertight approach is adopted for this site.

3.5.1 Watertight Option

A fully watertight basement approach may be adopted for this site. Grounded’s Hydrogeological Report (File No. 24-076) provides further discussion on this. A watertight basement implies that the basement structure is 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. 121.4 ±m. A connection to the City’s sewer for emergency repair services is recommended.

Although the City of Toronto is likely to require a watertight basement at this site (per the discussions in Section 3), a drained basement is also feasible from an engineering perspective. Grounded can provide recommendations for perimeter and subfloor drainage systems on request.

3.5.2 Drained Option

Per the discussion above, a drained basement approach at this site is feasible from a geotechnical engineering perspective if an exemption is granted by the City. The following discussion pertains to a drained basement approach only.

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. Perimeter drainage must be collected and conveyed directly to the building sumps, and not discharged into the subfloor drainage system, the granular layer, or beneath the floor slab.

Subfloor drainage pipes are to be spaced at a maximum 6 m (measured on-centres).

The walls of the substructure are to be fully drained to eliminate hydrostatic pressure. Where drained basement walls are made directly against shoring (or an existing foundation wall, as may be the case at this site), 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 drainage layer and the foundation wall should be considered to protect interior finishes from moisture.

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.

The permanent dewatering requirements are provided in Grounded’s Hydrogeological Report (File No. 24-076).

If any water is to be discharged to the storm or sanitary sewers, the City will require Discharge Agreements to be in place. Although a drained basement approach may be technically feasible, the City of Toronto will likely prohibit long-term discharge in light of their policy on Managing Foundation Drainage unless an exemption is granted.

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 wet upper sands are Type 4 soils, or Type 3 soils if dewatered
- The glacial till and silts and clays units are Type 2 soils

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, per Section 234:

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 239 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.

Excess soil is governed by Ontario Regulation 406/19: On-Site and Excess Soil Management (ESM). The Project Leader (typically the owner) may be required to file a notice in the excess soil registry and a Qualified Person (within the meaning of O.Reg. 153/04) may be required to prepare the associated planning documents and/or develop and implement a tracking system in accordance with the Soil Rules, to track each load of excess soil during its transportation and deposit before removing excess soil from the project area.

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.

The groundwater table at Elev. 121.4± m is above the bulk excavation level for P3. It is within the cohesionless upper sands unit as well as the underlying cohesive glacial till and silts and clays. While the cohesive soils will preclude the free-flow of groundwater, the upper sands will yield free-flowing water when penetrated.

For the upper sands, positive dewatering to lower the groundwater table may be required to facilitate construction as well as to maintain the integrity of the subgrade for foundation and slab-on-grade support. Alternatively, a full caisson cutoff wall could be considered to limit groundwater seepage from this soil unit. Other means and methods could be considered for controlling groundwater seepage from these soils, although anecdotally these soils have yielded consistent volumes of free-flowing groundwater at neighbouring sites. Using permeable shoring and sumps and pumps inside the excavation could lead to loss of ground issues.

Should positive dewatering be the preferred approach, dewatering will take some time to accomplish prior to the start of excavation. The water level should be kept at least 1.2 m below the lowest excavation elevation during construction, although it is acknowledged that dewatering the cohesive soils may not be feasible. However, the upper sands unit will require active groundwater control measures (or impermeable shoring).

Failure to dewater prior to excavation will result in unrecoverable disturbance of the subgrade, which will render advice provided for undisturbed subgrade conditions inapplicable.

A professional dewatering contractor should 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.

A watertight basement may be required. During construction, it will be necessary to consider the potential uplift pressure on the underside of a raft foundation due to hydrostatic forces. Positive dewatering operations during construction must begin prior to excavation and must continue until such time as the structural dead load exceeds the potential uplift forces (with suitable partial factors (LRFD) included in this assessment). A design groundwater elevation of 121.4 m is to be used in this assessment.

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

4.3 Earth-Retention Shoring Systems

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.

4.3.1 Lateral Earth Pressure Distribution

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

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

$\boldsymbol{P = 0.8\, K[\gamma H + q] + \gamma_w h_w}$ … in cohesive soils
$\boldsymbol{P = 0.65\, K[\gamma H + q] + \gamma_w h_w}$ … 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. 121.4 m must be accounted for.

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

4.3.2 Soldier Pile Toe Embedment

Soldier pile toes will be made in very stiff to hard cohesive soils. Soldier pile toes resist horizontal movement due to the passive earth pressure acting on the toe below the base of excavation.

Zones within the subgrade soils at this site are cohesionless, wet, and permeable. Augered holes for piles made into these soils will be prone to caving. 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 augered holes, mud/slurry/polymer drilling techniques, tremie pour concrete, or other methods as deemed necessary by the shoring contractor are required. Concrete for shoring piles and fillers must be placed by tremie method wherever there is more than 300 mm of water or fluid at the base of the drill hole.

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/hard subgrade, it is expected that post-grouted anchors can be made such that an anchor will safely carry up to 70 kN/m of adhered anchor length within in the sands, or 60 kN/m of adhered anchor length above Elev. 116 m, and 45kN/m below Elev. 116 m (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 stiff to hard subgrade below the proposed FFE is suitable for the placement of raker foundations. Raker footings established on very stiff soils at an inclination of 45 degrees can be designed for a maximum factored geotechnical resistance at ULS of 250 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 excess 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 preliminary 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 foundation installations and 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.

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 should be monitored by Grounded at the time of construction to confirm material quality, and thickness.

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

Since deep drilling is excluded from the current scope of work, this geotechnical engineering report provides preliminary recommendations for the following elements:

- Raft design (deep boreholes with pressuremeter needed)
- Caisson design (deep boreholes with rock coring needed)

Once the preferred foundation system is known at detailed design, additional boreholes and updated detailed geotechnical engineering advice are required to confirm each of the elements listed above. Once completed, a future geotechnical engineering report by Grounded Engineering would then supersede this report. Note that preliminary findings can vary significantly from the findings of a detailed comprehensive study.

Grounded should be retained to review the structural and geostructural engineering drawings prior to issue or construction to ensure that the recommendations in this report have been appropriately implemented.

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 Grounded’s standard of practice as well as other reasonable and prudent 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 to potential site alteration.

This report provides preliminary geotechnical engineering advice intended for use by the owner and their retained design team. These preliminary interpretations, design parameters, advice, and discussion on construction considerations are not complete. A detailed site-specific geotechnical investigation must be conducted by Grounded during detailed design to confirm and update the preliminary recommendations provided here.

5.3 Report Use

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

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

6 Closure

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

For and on behalf of our team,

GROUNDED
ENGINEERING

A.K

Andrew Kernerman, B.A.Sc., EIT.
Project Coordinator

LICENSED PROFESSIONAL ENGINEER
M. DIEZ DE AUX
100159147
06/08/2024
PROVINCE OF ONTARIO

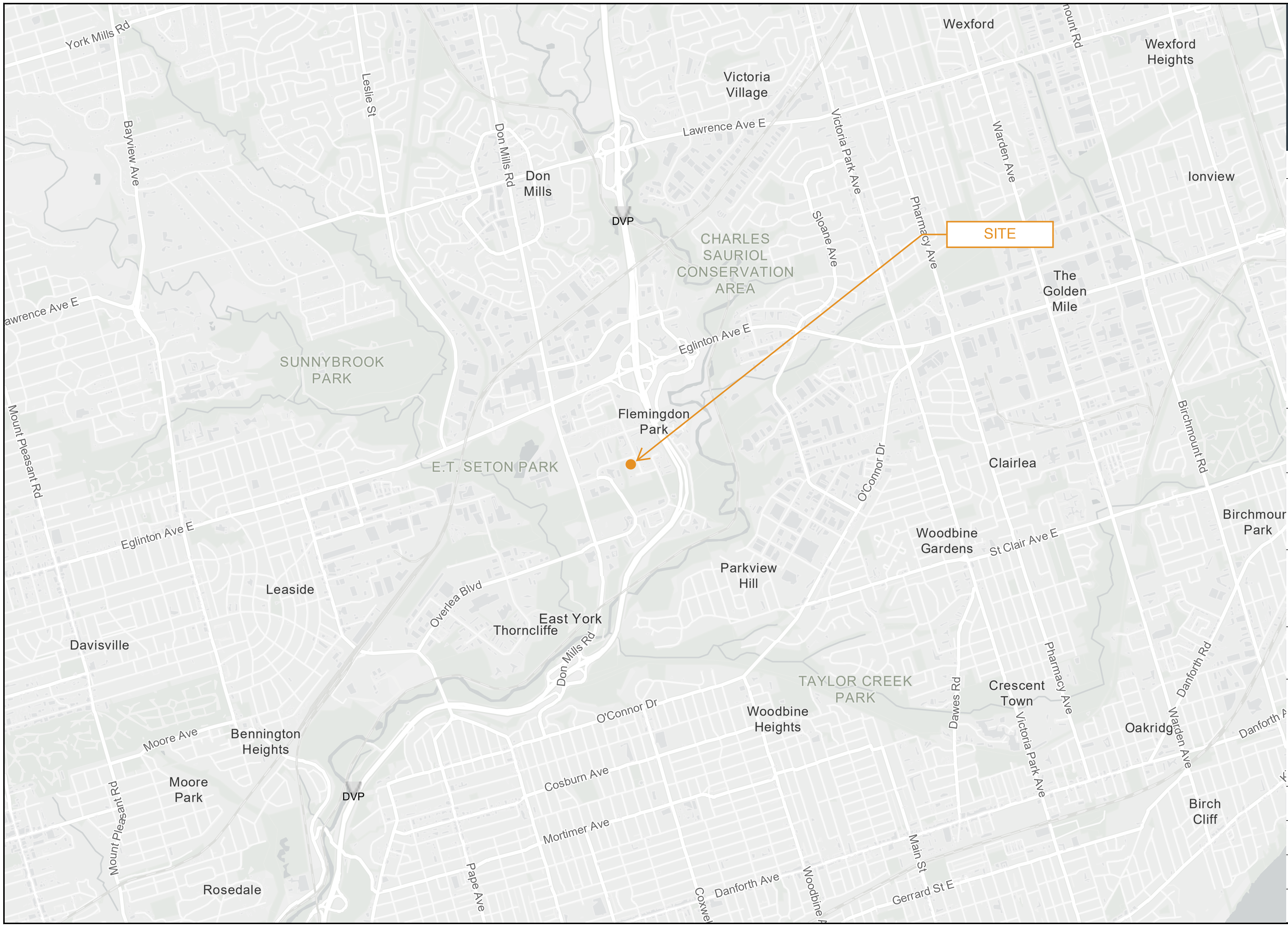
diez

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

LICENSED PROFESSIONAL ENGINEER
J. J. CROWDER
100077148
2024-08-08
PROVINCE OF ONTARIO

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

FIGURES



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LEGEND

● APPROXIMATE SITE LOCATION

Note

Reference
ArcGIS Online 2024

Project
45 GRENOBLE DRIVE, TORONTO, ONTARIO

Figure Title
SITE LOCATION PLAN

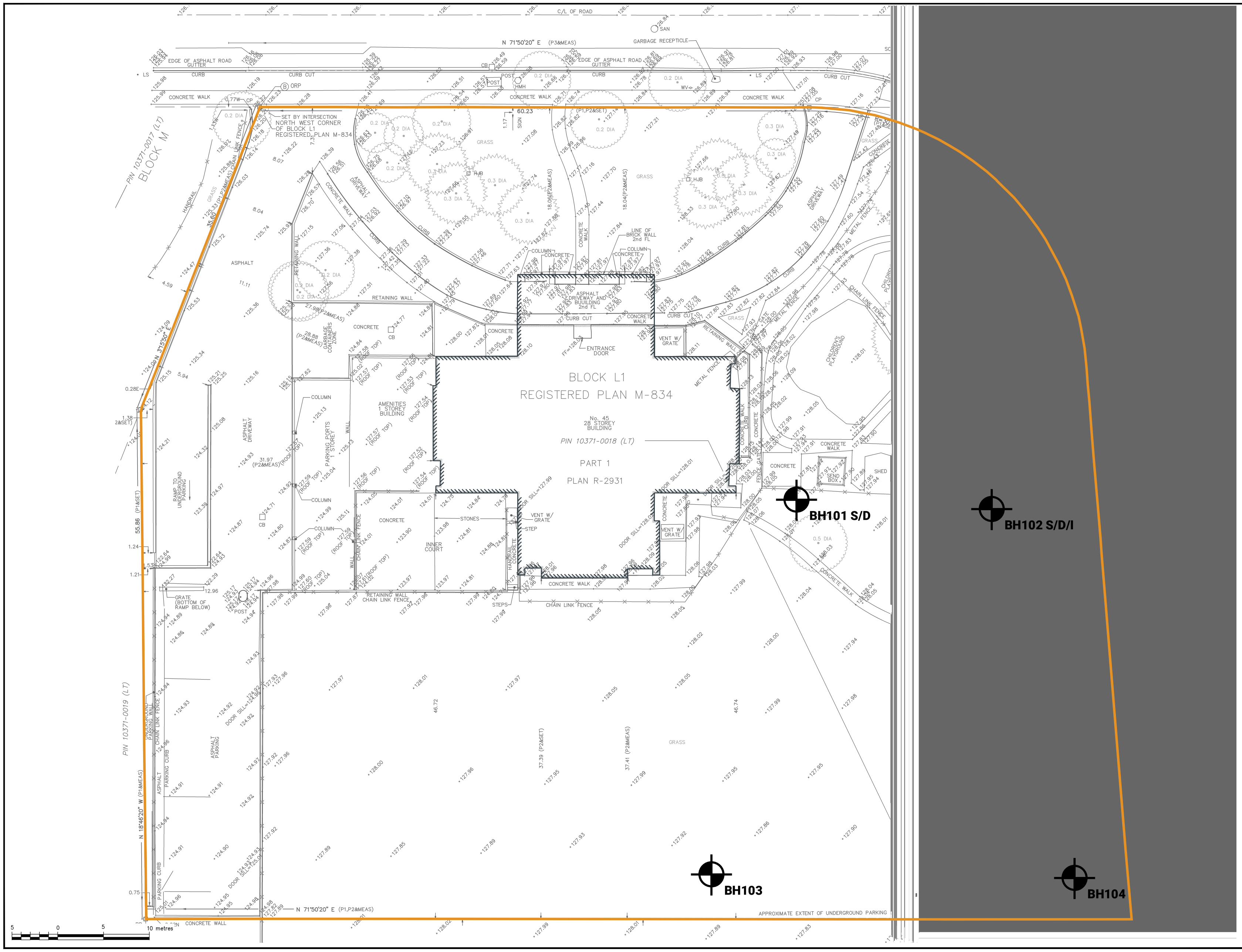
North
TRUE
PROJECT

Date
JULY 2024

Scale
NTS

Job No
24-076

Figure No
FIGURE 1



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LEGEND

—	PROPERTY BOUNDARY
▨	EXISTING BUILDING STRUCTURE
⊕	MONITORING WELL/BOREHOLE BY GROUNDED

Note

Reference

Site survey prepared by JD Barnes
Dated March 20, 2023

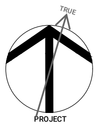
Project

45 GRENOBLE DRIVE
TORONTO, ONTARIO

Figure Title

BOREHOLE LOCATION PLAN
EXISTING CONDITIONS

North



Date

JULY 2024

Scale

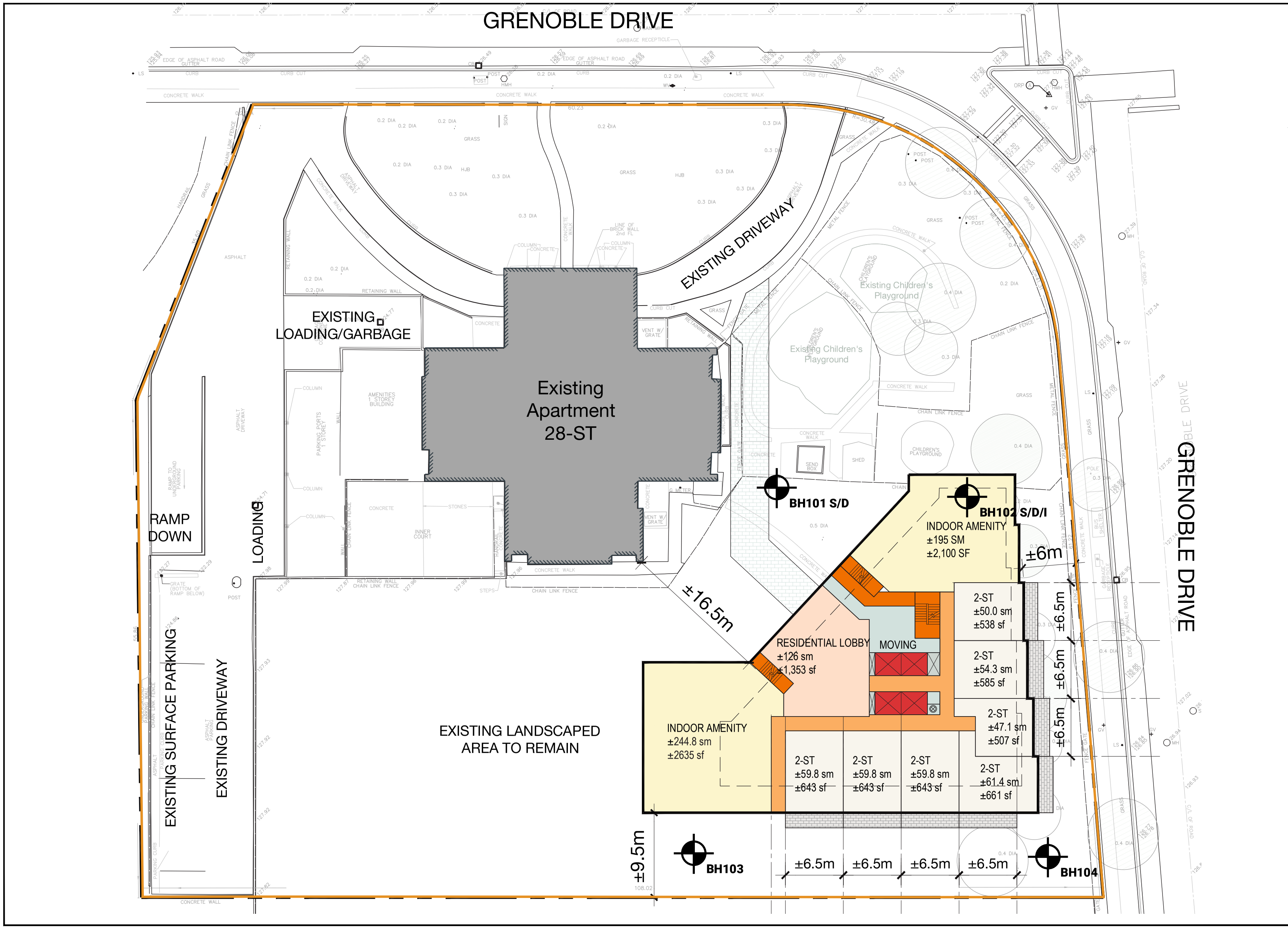
AS INDICATED

Job No

24-076

Figure No

FIGURE 2



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LEGEND

- PROPERTY BOUNDARY
- EXISTING BUILDING STRUCTURE
- MONITORING WELL/BOREHOLE BY GROUNDED

Note

Reference

Architectural Drawings prepared by BDP Quadrangle Ltd.

Dated May 22, 2024

Project

**45 GRENOBLE DRIVE
TORONTO, ONTARIO**

Figure Title

**BOREHOLE LOCATION PLAN
PROPOSED CONDITIONS**

North

Date

JULY 2024

Scale

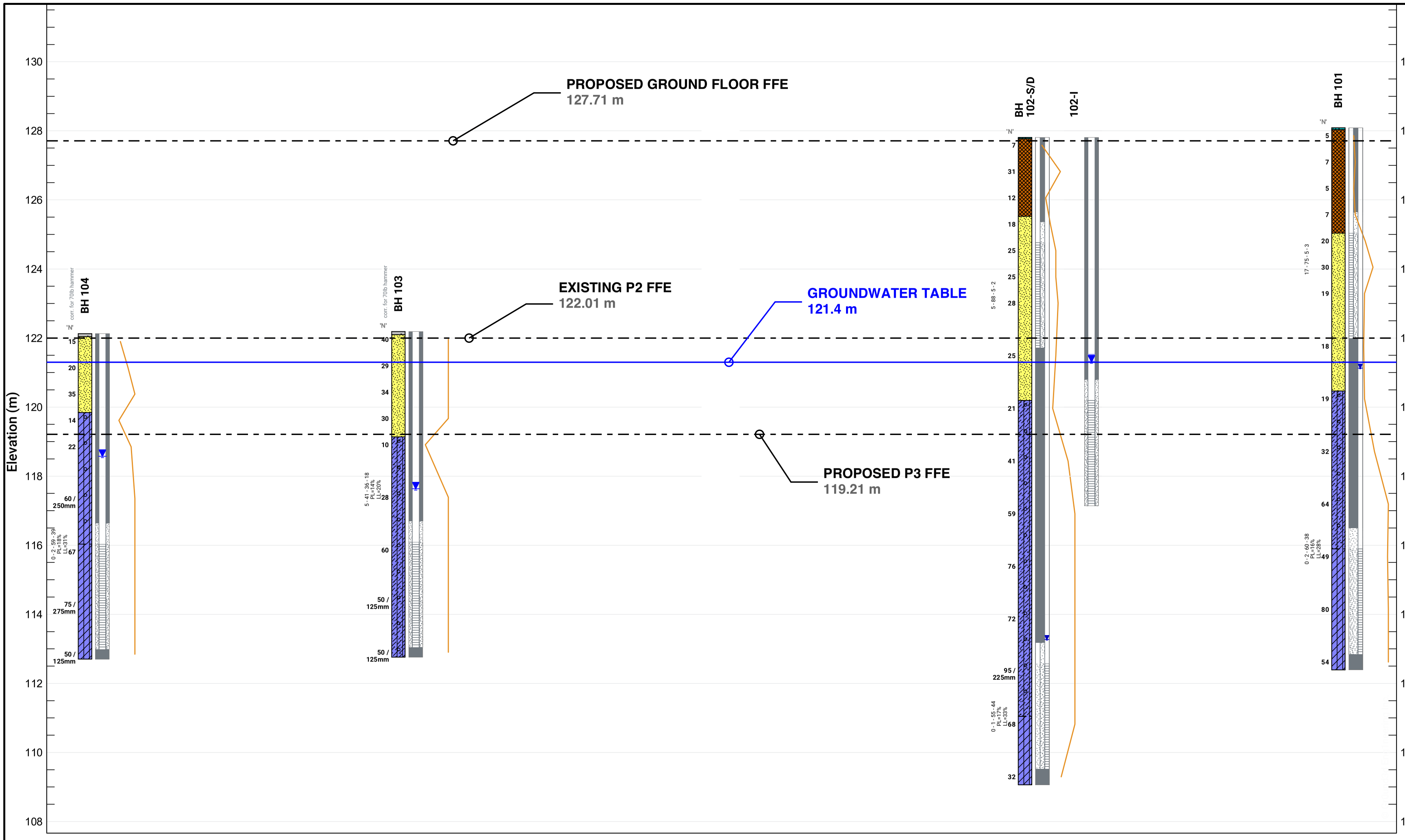
0m 6.25m 12.5m

Job No

24-076

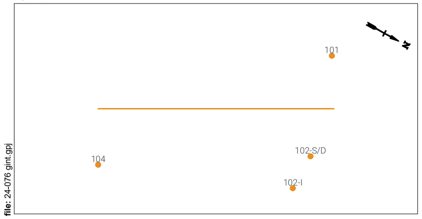
Figure No

FIGURE 3



Boreholes Equally Spaced

SITE MAP



BOREHOLE STRATIGRAPHY LEGEND

- Topsoil
- Fill
- Sand
- Clayey Silt Till
- Silt and Clay
- Concrete
- Clay and Silt Till

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LEGEND

🟧	FILL
🟧	GRAVELS (gravel to gravelly sand)
🟨	SILT TO SAND (not till)
🟩	COHESIONLESS TILLS
🟦	COHESIVE SOILS (clayey silt to clay, incl. tills)
🟦	DISTURBED/REWORKED/ORGANIC
BH 101	BOREHOLES BY GROUNDED
T-BH7	BOREHOLES BY OTHERS

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

Project
45 GRENOBLE DR
TORONTO, ON

Figure Title
SUBSURFACE PROFILE

Date	AUGUST 2024
Scale	AS INDICATED
Job No	24-076
Figure No	**FIGURE 4**

APPENDIX A

BOREHOLE LOG TERMINOLOGY

GROUNDED ENGINEERING

SAMPLING/TESTING METHODS

SS: split spoon sample

AS: auger sample

GS: grab sample

FV: shear vane

DP: direct push

PMT: pressuremeter test

ST: shelby tube

CORE: soil coring

RUN: rock coring

SYMBOLS & ABBREVIATIONS

MC: moisture content

LL: liquid limit

PL: plastic limit

NP: non-plastic

γ: soil unit weight (bulk)

G_s: specific gravity

S_u: undrained shear strength

▽ unstabilized water level

▼ 1st water level measurement

▼ 2nd water level measurement most recent

▼ water level measurement

ENVIRONMENTAL SAMPLES

M&I: metals and inorganic parameters

PAH: polycyclic aromatic hydrocarbon

PCB: polychlorinated biphenyl

VOC: volatile organic compound

PHC: petroleum hydrocarbon

BTEX: benzene, toluene, ethylbenzene and xylene

PPM: parts per million

FIELD MOISTURE (based on tactile inspection)

DRY: no observable pore water

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

WET: visible pore water

COHESIONLESS

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

COHESIVE

Consistency	N-Value	Su (kPa)
Very Soft	<2	<12
Soft	2 - 4	12 - 25
Firm	4 - 8	25 - 50
Stiff	8 - 15	50 - 100
Very Stiff	15 - 30	100 - 200
Hard	>30	>200

COMPOSITION

Term	% by weight
trace silt	<10
some silt	10 - 20
silt***y***	20 - 35
sand ***and*** silt	>35

ASTM STANDARDS

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

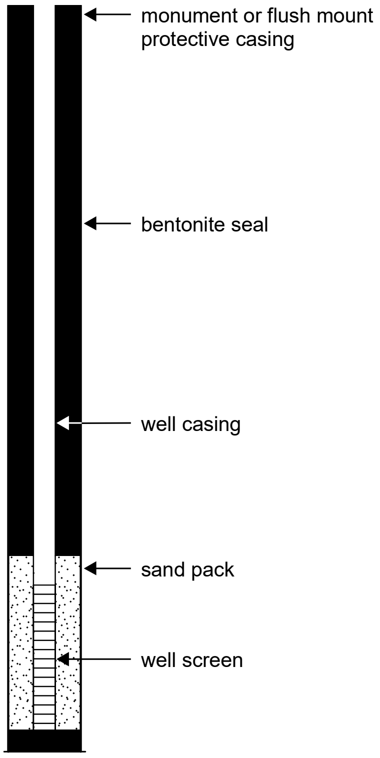
ASTM D3441 Cone Penetration Test (CPT)
Pushing an internal still rod with a outer hollow rod ("sleeve") tipped with a cone with an apex angle of 60° and a cross-sectional area of 1000 mm^2 into soil. The resistance is measured in the sleeve and at the tip to determine the skin friction and the tip resistance.

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

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

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

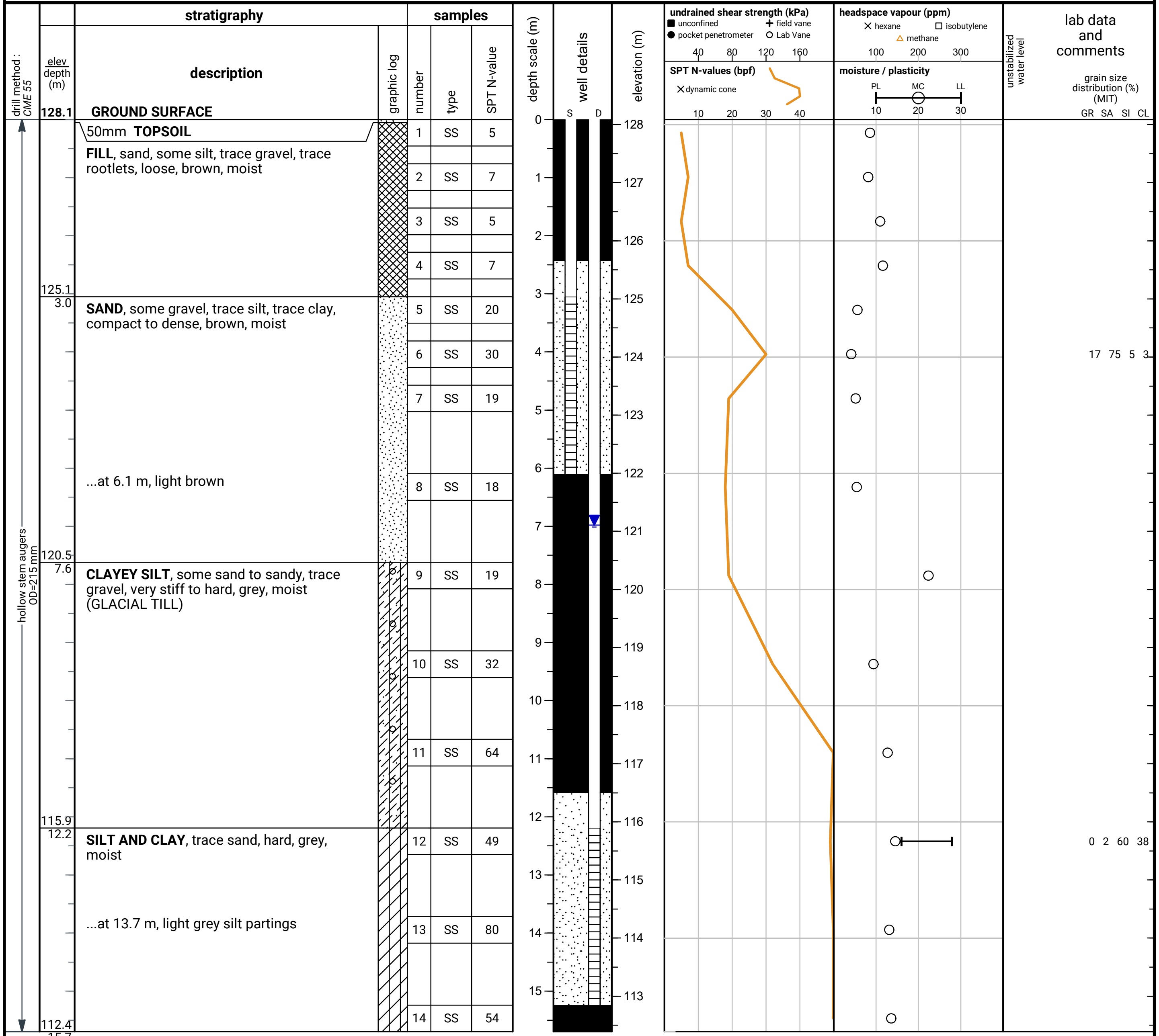
WELL LEGEND



Date Started : May 29, 2024
Position : E: 634482, N: 4841687 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 101

File No. : 24-076
Project : 45 Grenoble Dr, Toronto, ON
Client : Gateway Properties



elev depth (m)	description
128.1	**GROUND SURFACE**
	50mm **TOPSOIL**
	FILL, sand, some silt, trace gravel, trace rootlets, loose, brown, moist
125.1 3.0	**SAND**, some gravel, trace silt, trace clay, compact to dense, brown, moist ...at 6.1 m, light brown
120.5 7.6	**CLAYEY SILT**, some sand to sandy, trace gravel, very stiff to hard, grey, moist (GLACIAL TILL)
115.9 12.2	**SILT AND CLAY**, trace sand, hard, grey, moist ...at 13.7 m, light grey silt partings
112.4 15.7	

number	type	SPT N-value	GR SA SI CL
1	SS	5	
2	SS	7	
3	SS	5	
4	SS	7	
5	SS	20	
6	SS	30	17 75 5 3
7	SS	19	
8	SS	18	
9	SS	19	
10	SS	32	
11	SS	64	
12	SS	49	0 2 60 38
13	SS	80	
14	SS	54	

END OF BOREHOLE

Borehole was dry upon completion of drilling.

S: 50 mm dia. monitoring well installed.
D: 50 mm dia. monitoring well installed.
No. 10 screen

101-S GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jun 6, 2024	dry	n/a
Jun 20, 2024	dry	n/a
Jul 5, 2024	dry	n/a
Jul 19, 2024	dry	n/a

101-D GROUNDWATER LEVELS

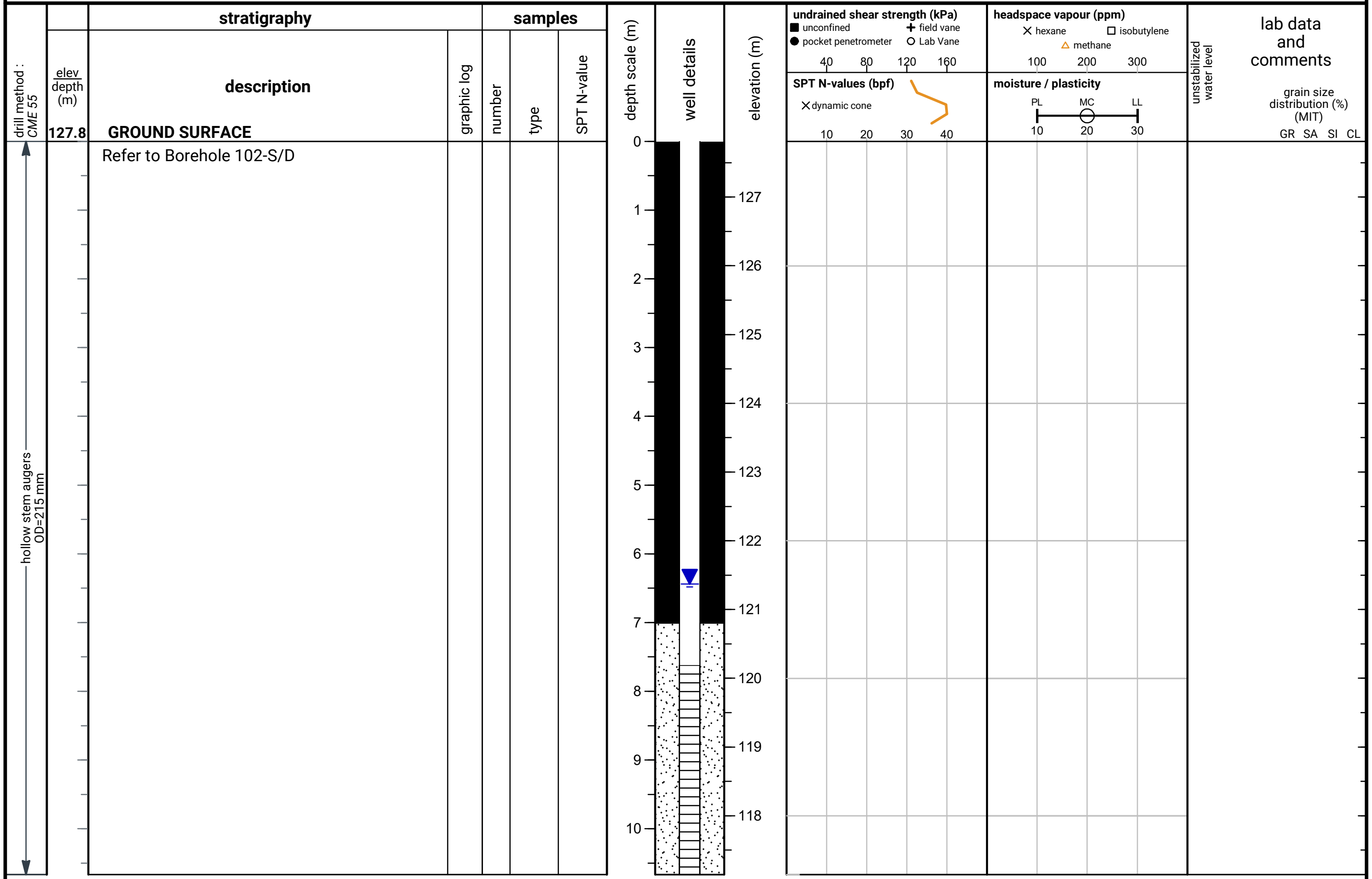
date	depth (m)	elevation (m)
Jun 6, 2024	7.0	121.1
Jun 20, 2024	7.0	121.1
Jul 5, 2024	7.0	121.1
Jul 19, 2024	7.0	121.1

GROUNDED ENGINEERING

Date Started : May 27, 2024
Position : E: 634511, N: 4841693 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 102-I

File No. : 24-076 **Project** : 45 Grenoble Dr, Toronto, ON **Client** : Gateway Properties



END OF BOREHOLE

Borehole was dry upon completion of drilling.

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

GROUNDWATER LEVELS

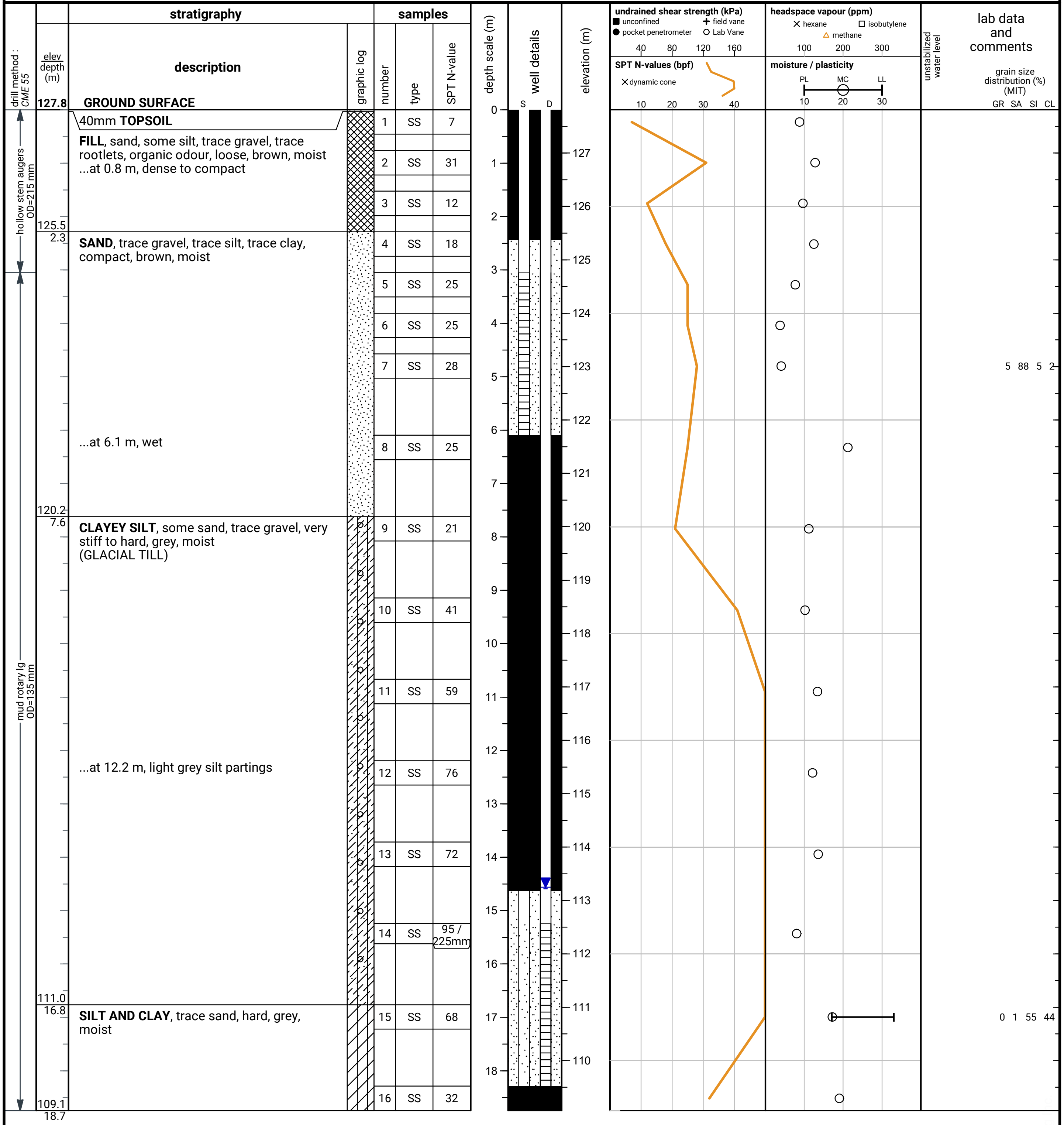
date	depth (m)	elevation (m)
Jun 6, 2024	6.5	121.3
Jun 20, 2024	6.5	121.3
Jul 5, 2024	6.5	121.3
Jul 19, 2024	6.4	121.4

file: 24-076 gint.gpj

Date Started : May 27, 2024
Position : E: 634503, N: 4841693 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 102-S/D

File No. : 24-076 **Project** : 45 Grenoble Dr, Toronto, ON **Client** : Gateway Properties



elev depth (m)	description	number	type	SPT N-value
127.8	**GROUND SURFACE**			
	40mm **TOPSOIL**	1	SS	7
	FILL, sand, some silt, trace gravel, trace rootlets, organic odour, loose, brown, moist ...at 0.8 m, dense to compact	2	SS	31
		3	SS	12
125.5 2.3	**SAND**, trace gravel, trace silt, trace clay, compact, brown, moist	4	SS	18
		5	SS	25
		6	SS	25
		7	SS	28
	...at 6.1 m, wet	8	SS	25
120.2 7.6	**CLAYEY SILT**, some sand, trace gravel, very stiff to hard, grey, moist (GLACIAL TILL)	9	SS	21
		10	SS	41
		11	SS	59
	...at 12.2 m, light grey silt partings	12	SS	76
		13	SS	72
		14	SS	95 / 225mm
111.0 16.8	**SILT AND CLAY**, trace sand, hard, grey, moist	15	SS	68
109.1 18.7		16	SS	32

Sample	GR	SA	SI	CL
7	5	88	5	2
15	0	1	55	44

END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

S: 50 mm dia. monitoring well installed.
D: 50 mm dia. monitoring well installed.
No. 10 screen

102-S/D-S GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jun 6, 2024	dry	n/a
Jun 20, 2024	dry	n/a
Jul 5, 2024	dry	n/a
Jul 19, 2024	dry	n/a

102-S/D-D GROUNDWATER LEVELS

date	depth (m)	elevation (m)
Jun 6, 2024	17.2	110.6
Jun 20, 2024	15.9	111.9
Jul 5, 2024	15.2	112.6
Jul 19, 2024	14.6	113.2

file: 24-076 gint.gpj

Tech : NA | **PM** : AK | **Rev** : MD

GROUNDED ENGINEERING

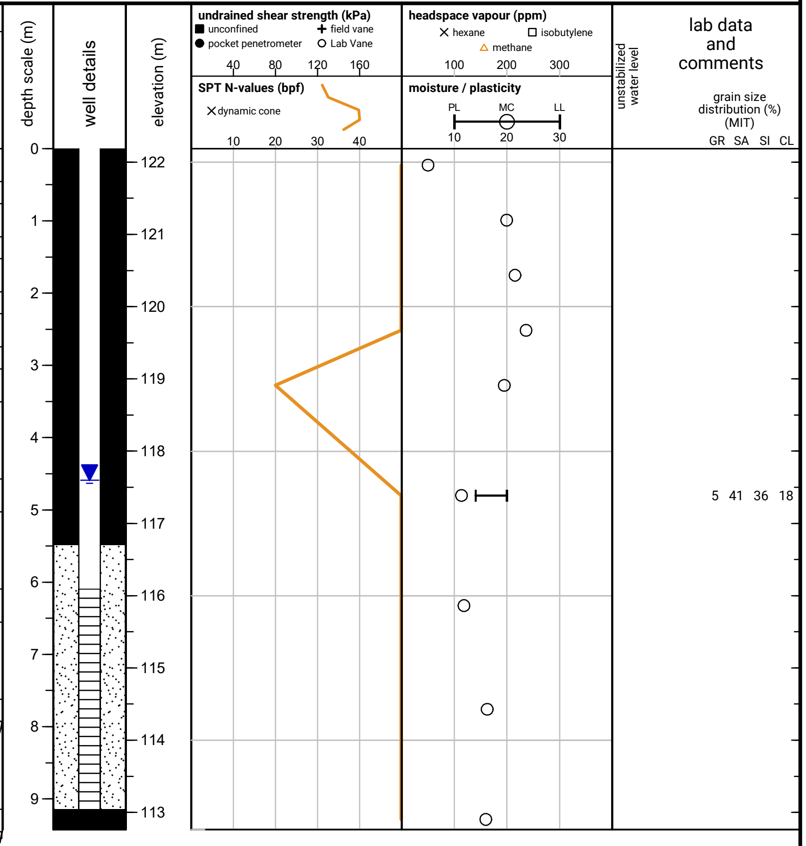
Date Started : May 29, 2024
Position : E: 634488, N: 4841641 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 103

File No. : 24-076 **Project** : 45 Grenoble Dr, Toronto, ON **Client** : Gateway Properties

drill method : *Hilti DD350*

drill method	elev depth (m)	stratigraphy description	graphic log	sample number	type	SPT N-value*
	122.2	**TOP OF SLAB**				
hollow stem augers OD=215 mm		90mm **CONCRETE**		1	SS	40
		SAND, trace silt, trace gravel, compact to dense, brown, moist ...at 0.8 m, wet		2	SS	29
				3	SS	34
				4	SS	30
mud rotary lg OD=135 mm	119.2 3.0	**CLAYEY SILT**, some sand, trace gravel, stiff to very stiff, grey, moist (GLACIAL TILL)		5	SS	10
		...at 4.6 m, seam of sand and silt		6	SS	28
		...at 6.1 m, hard		7	SS	60
				8	SS	50 / 125mm
	112.8 9.4			9	SS	50 / 125mm



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)
Jun 6, 2024	7.4	114.8
Jun 20, 2024	6.8	115.4
Jul 5, 2024	5.5	116.7
Jul 19, 2024	4.6	117.6

file: 24-076 gint.gpj

Tech : IH | **PM** : AK | **Rev** : MD

* SPT N-values corrected based on energy of 32 kg hammer dropped 760 mm

GROUNDED
ENGINEERING

Date Started : May 27, 2024
Position : E: 634526, N: 4841654 (UTM 17T)
Elev. Datum : Geodetic

BOREHOLE LOG 104

File No. : 24-076

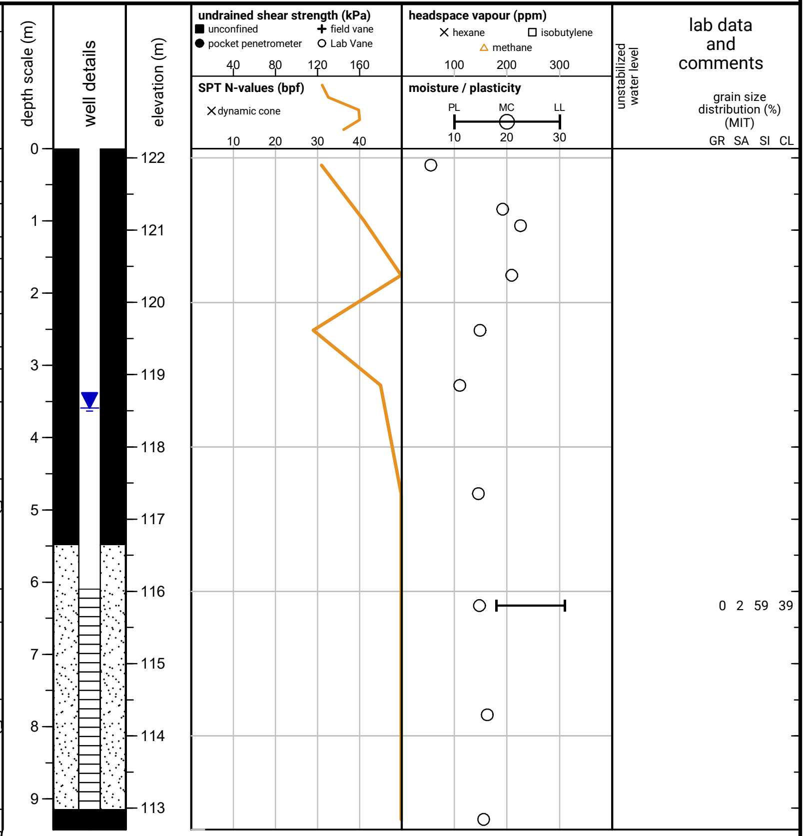
Project : 45 Grenoble Dr, Toronto, ON

Client : Gateway Properties

drill method : *Hilti DD350*

drill method	elev depth (m)	description (stratigraphy)
	122.1	**TOP OF SLAB**
hollow stem augers OD=215 mm		90mm **CONCRETE**
		SAND, trace silt, trace gravel, compact to dense, brown, moist ...at 0.8 m, wet
mud rotary lg OD=135 mm	119.8 2.3	**CLAY AND SILT**, some sand, trace gravel, stiff to very stiff, grey, moist (GLACIAL TILL) ...at 4.6 m, hard
	116.0 6.1	**SILT AND CLAY**, trace sand, hard, grey, moist
	112.7 9.4	**END OF BOREHOLE**

number	type	SPT N-value*
1	SS	15
2A 2B	SS	20
3	SS	35
4	SS	14
5	SS	22
6	SS	60 / 250mm
7	SS	67
8	SS	75 / 275mm
9	SS	50 / 125mm



END OF BOREHOLE

Borehole was filled with drill water upon completion of drilling.

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

GROUNDWATER LEVELS

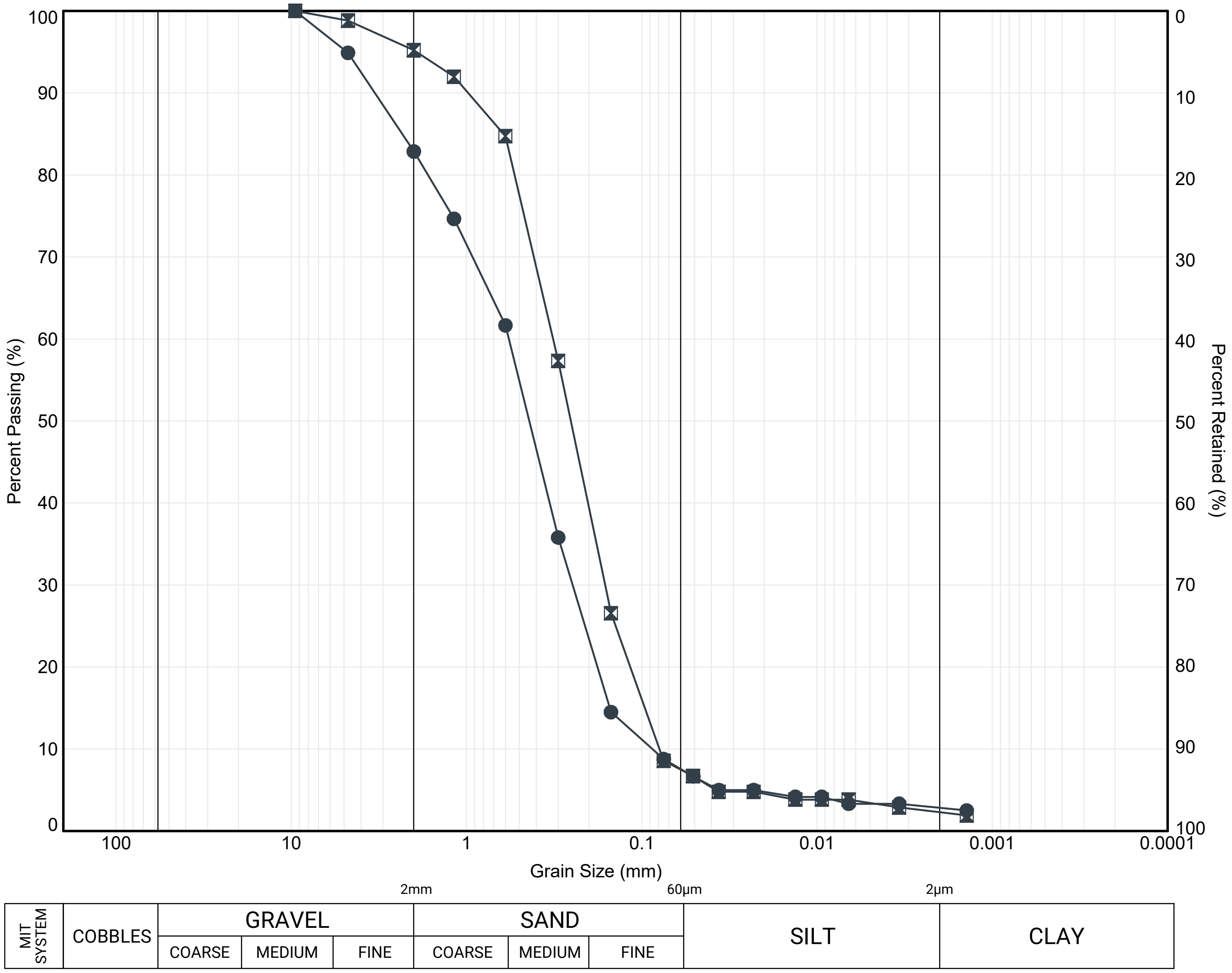
date	depth (m)	elevation (m)
Jun 6, 2024	7.0	115.1
Jun 20, 2024	5.9	116.2
Jul 5, 2024	4.2	117.9
Jul 19, 2024	3.6	118.5

file: 24-076 gint.gpj

Tech : IH | **PM** : AK | **Rev** : MD

* SPT N-values corrected based on energy of 32 kg hammer dropped 760 mm

APPENDIX B



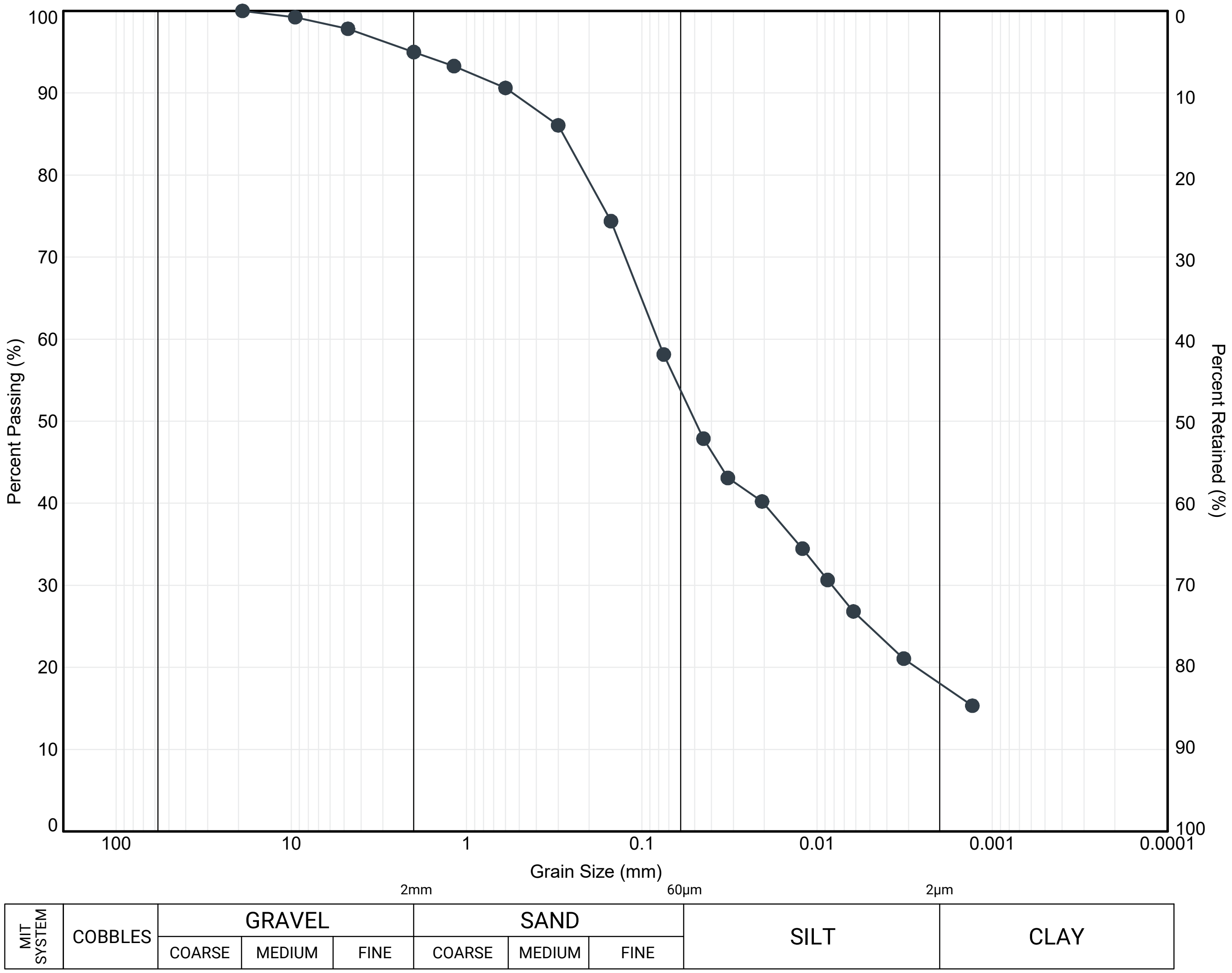
					MIT SYSTEM			
	Location	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**BH 101**	SS6	4.0	124.0	17	75	5	3
☒	**BH 102-S/D**	SS7	4.8	123.0	5	88	5	2

GROUNDED
ENGINEERING

Title: **GRAIN SIZE DISTRIBUTION**
SANDS

File No.: **24-076**

file: 24-076 gint.gpj

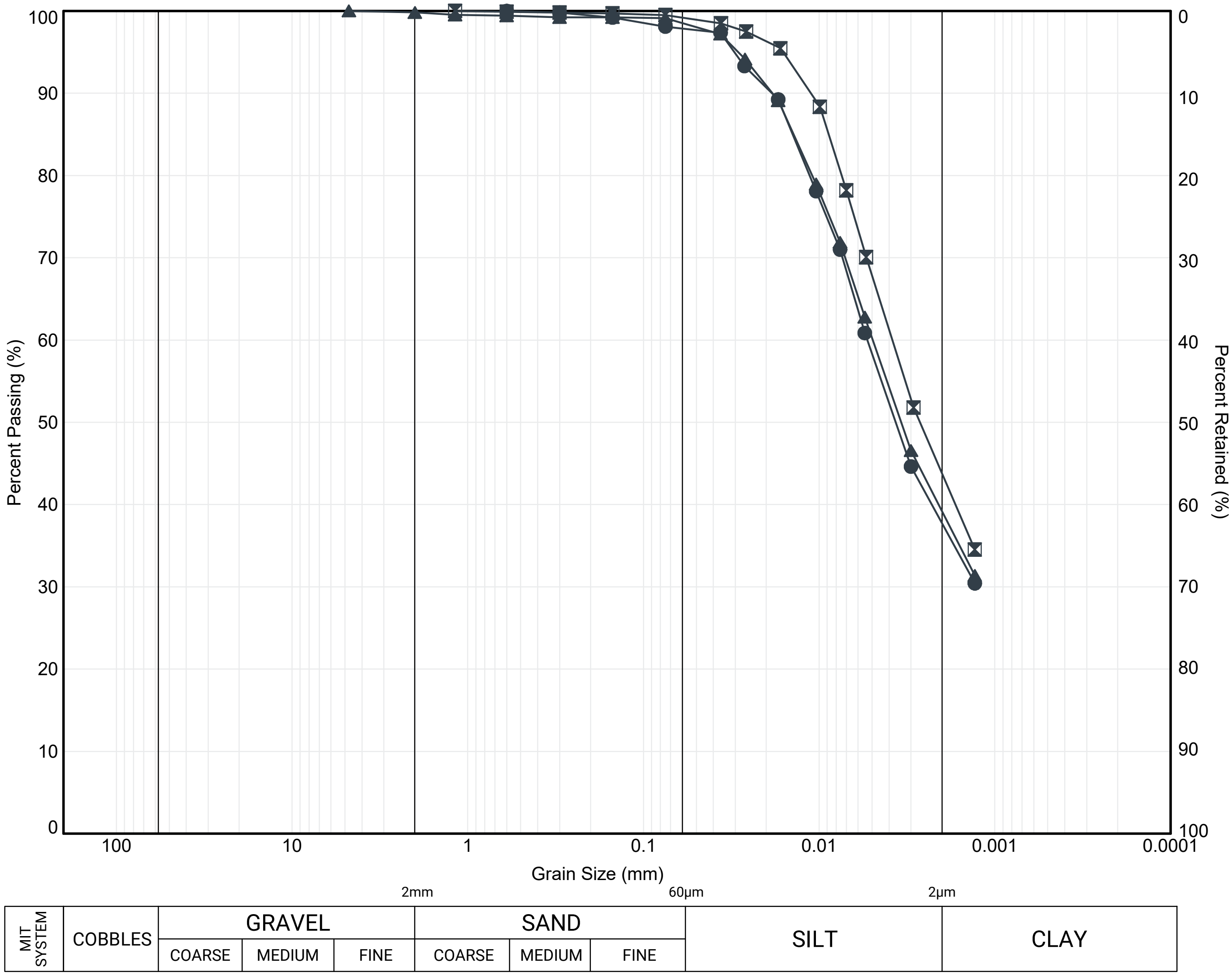


					MIT SYSTEM			
	Location	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**BH 103**	SS6	4.8	117.4	5	41	36	18

GROUNDED
ENGINEERING

Title: **GRAIN SIZE DISTRIBUTION**
GLACIAL TILL

File No.: **24-076**



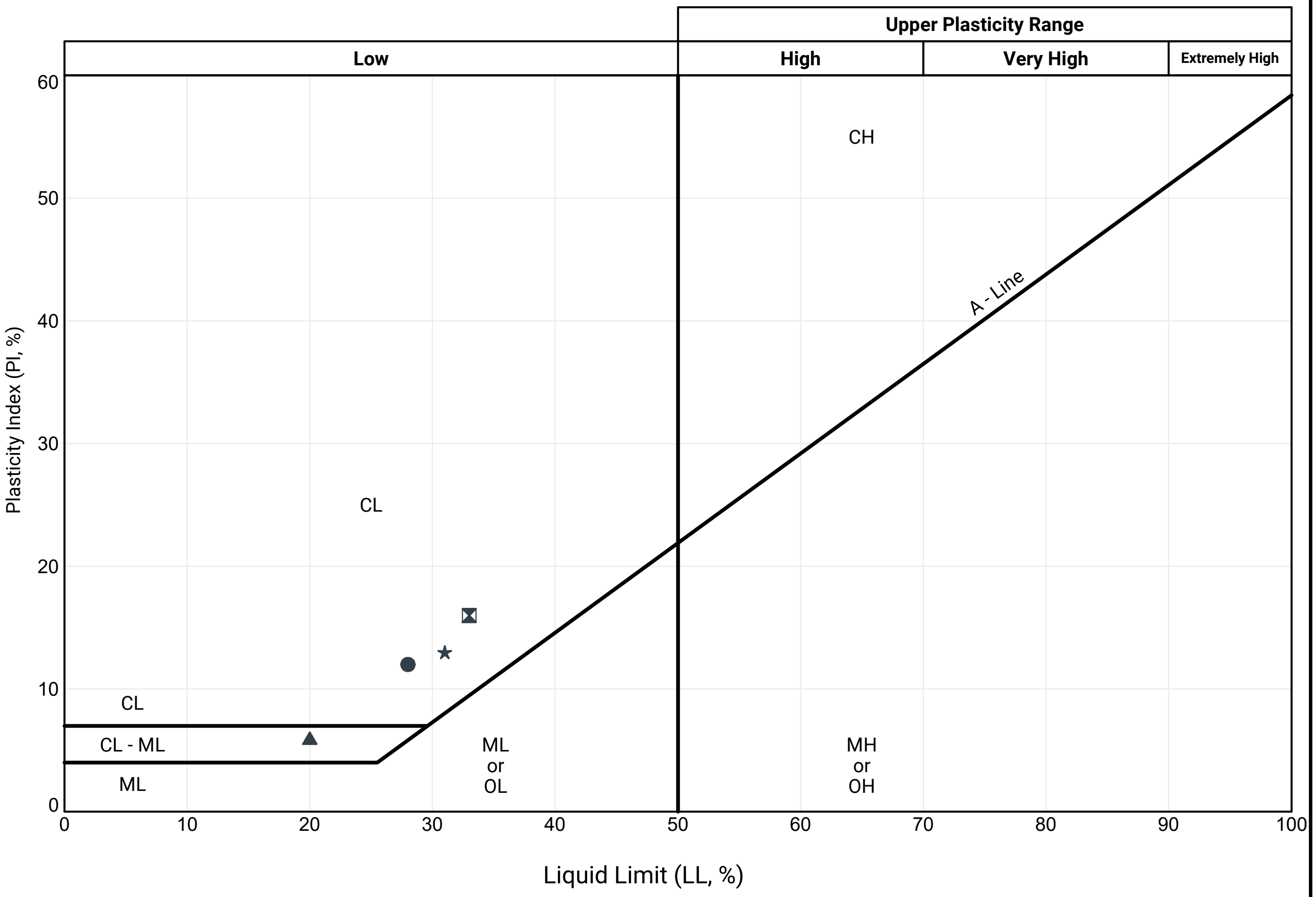
MIT SYSTEM

	Location	Sample	Depth (m)	Elev. (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
●	**BH 101**	SS12	12.4	115.7	0	2	60	38
☒	**BH 102-S/D**	SS15	17.0	110.8	0	1	55	44
▲	**BH 104**	SS7	6.3	115.8	0	2	59	39

GROUNDED
ENGINEERING

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

File No.: **24-076**



	Location	Sample	Depth (m)	Elev. (m)	LL (%)	PL (%)	PI (%)
●	**BH 101**	SS12	12.4	115.7	28	16	12
☒	**BH 102-S/D**	SS15	17.0	110.8	33	17	16
▲	**BH 103**	SS6	4.8	117.4	20	14	6
★	**BH 104**	SS7	6.3	115.8	31	18	13

file: 24-076 gint.gpj

GROUNDED ENGINEERING

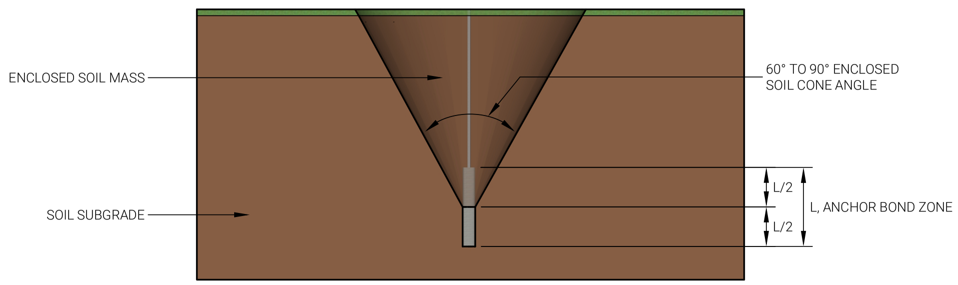
Title:

ATTERBERG LIMITS CHART

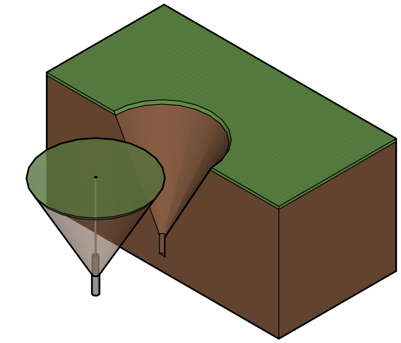
File No.: **24-076**

APPENDIX C

INDIVIDUAL SOIL TIEDOWN ANCHOR

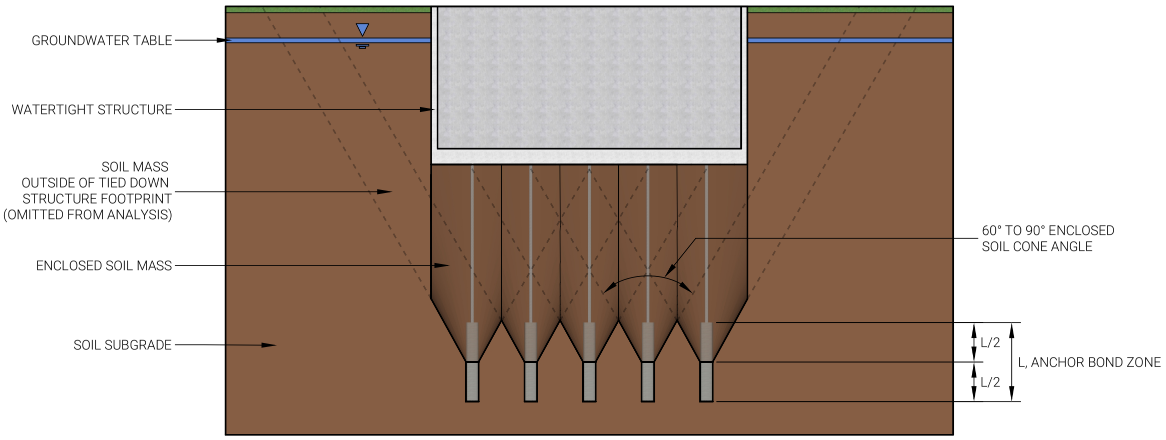


SECTIONAL VIEW

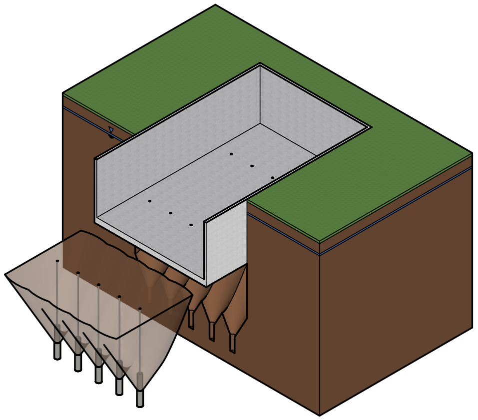


ISOMETRIC VIEW

GRID OF SOIL TIEDOWN ANCHORS WITH STRUCTURE



SECTIONAL VIEW



ISOMETRIC VIEW

NOTES:

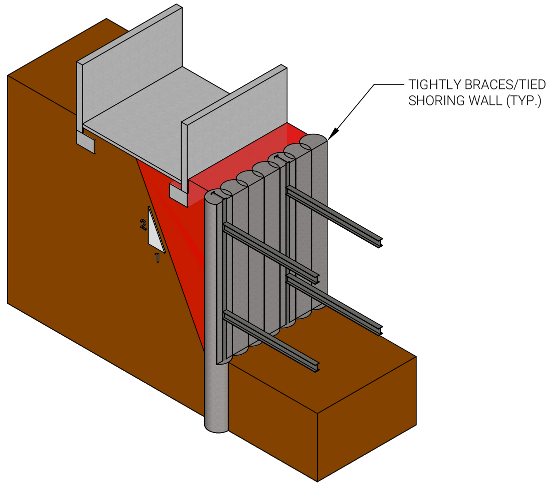
1. UNFACTORED EQUILIBRIUM BETWEEN A STRUCTURE AND UPLIFT IS ESTABLISHED WHEN THE TOTAL WEIGHT OF THE STRUCTURE AND THE EFFECTIVE WEIGHT (CALCULATED USING γ') OF THE ENCLOSED SOIL MASS BELOW THE STRUCTURE IS EQUAL TO THE TOTAL UPLIFT PRESSURE (FHWA GEOTECHNICAL ENGINEERING CIRCULAR NO. 4 - GROUND ANCHORS AND ANCHORED SYSTEMS, 1999).
2. THE WEIGHT OF OVERLAPPING ENCLOSED SOIL MASSES MUST ONLY BE ACCOUNTED FOR ONCE.
3. THE WEIGHT OF SOIL OUTSIDE OF THE FOOTPRINT OF THE TIED DOWN STRUCTURE SHOULD BE NEGLECTED.

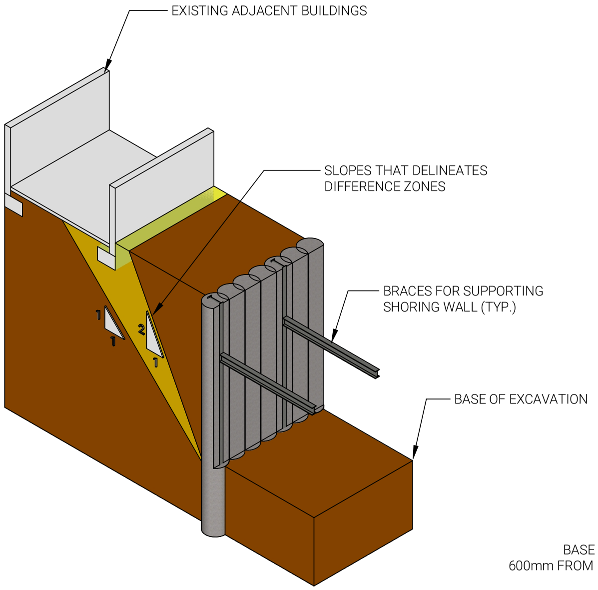
NOT TO SCALE. FEATURES ARE EXAGGERATED FOR DEMOSTRATION PURPOSES.

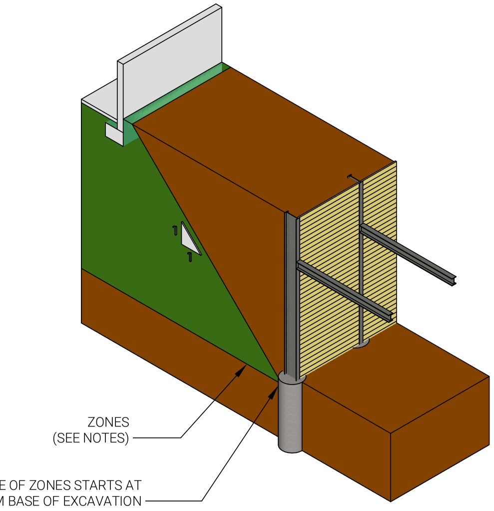
GROUNDED
ENGINEERING

Title

SOIL TIEDOWN ANCHOR GLOBAL STABILITY







ZONE A (RED)

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

ZONE B (YELLOW)

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

ZONE C (GREEN)

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

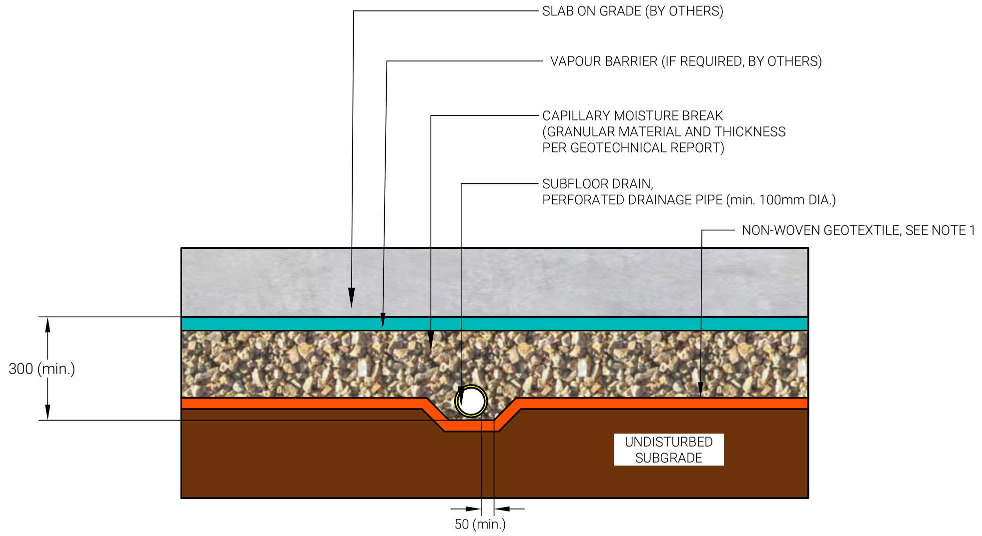
NOTES:
1. USER'S GUIDE - NBC 2005 STRUCTURAL COMMENTARIES (PART 4 OF DIVISION B) - COMMENTARY K.

GROUNDED ENGINEERING

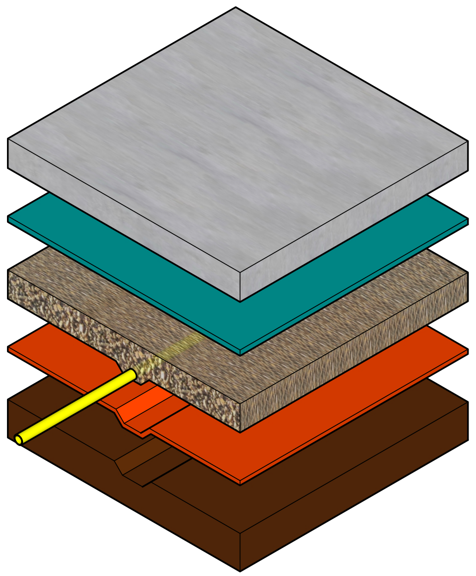
Title

EXCAVATION ZONE OF INFLUENCE GUIDELINES

OBJECTS ARE COLOR-CODED BETWEEN TWO VIEWS FOR CLARITY



SECTIONAL VIEW



ISOMETRIC VIEW

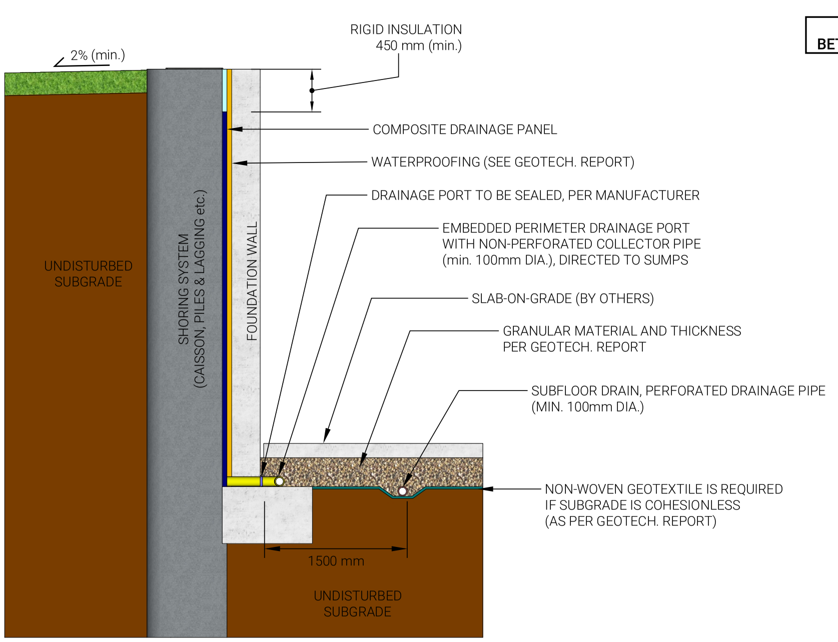
NOTES

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

GROUNDED
ENGINEERING

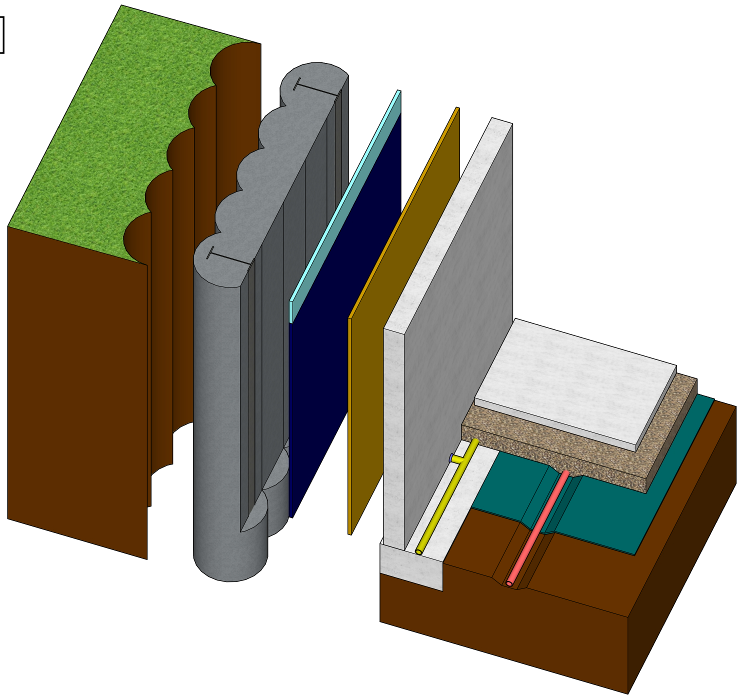
Title

BASEMENT SUBDRAIN TYPICAL DETAIL



SECTIONAL VIEW

**OBJECTS ARE COLOR-CODED
BETWEEN TWO VIEWS FOR CLARITY**



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. SEE THE GEOTECHNICAL REPORT FOR GEOTEXTILE REQUIREMENTS.

PERIMETER DRAINAGE SYSTEM

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

GENERAL NOTES

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

GROUNDED
ENGINEERING

Title

BASEMENT DRAINAGE SHORING SYSTEM TYPICAL DETAILS