



**REPORT ON  
GEOTECHNICAL INVESTIGATION  
115 WATSON PARKWAY NORTH  
GUELPH, ONTARIO**

**REPORT NO.: 4515-22-GC  
REPORT DATE: JUNE 7, 2022**

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# Table of Contents

**1.0 INTRODUCTION.....1**

**2.0 SITE CONDITION.....1**

**3.0 INVESTIGATION PROCEDURE.....2**

**4.0 SUMMARIZED SITE AND SUBSURFACE CONDITIONS.....3**

**4.1 Surface Course.....3**

**4.2 Fill.....3**

**4.3 Sand and Gravel.....4**

**4.4 Sandy Silt Till.....4**

**4.5 Silty Sand.....5**

**4.6 Groundwater.....5**

**5.0 RECOMMENDATIONS.....7**

**5.1 Site Preparation.....7**

**5.2 Foundation Design.....8**

**5.3 Floor Slab Construction.....10**

**5.4 Earthquake Consideration.....11**

**5.5 Excavation and Backfilling.....11**

**5.6 Lateral Earth Pressure.....12**

**5.7 Permanent Perimeter Drainage.....13**

**5.8 Groundwater Control.....13**

**5.9 Pavement Construction.....13**

**6.0 GENERAL STATEMENT OF LIMITATION.....15**

## DRAWINGS & FIGURES

<b>Borehole Location Plan</b>	<b>Drawing No.: 1</b>
<b>Borehole Logs (22BH-1 to 22BH-14)</b>	<b>Drawing Nos.: 2-17</b>
<b>Sections</b>	<b>Drawing Nos.: 18-20</b>
<b>Gradation Curve</b>	<b>Figure No.: 1</b>
<b>Permanent Perimeter Drainage System</b>	<b>Figure No.: 2</b>
<b>Suggested Backfill and Drainage System</b>	<b>Figure No. 3</b>

## APPENDIX A

<b>Shoring Design</b>	<b>Drawing No. A1</b>
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## 1.0 INTRODUCTION

*Toronto Inspection Ltd.* was retained by Tercot Communities to conduct an additional geotechnical investigation at a property, located at 115 Watson Parkway North (& Starwood Drive) in Guelph, Ontario (hereafter described as “the Site”). The field work of the current investigation was carried out in conjunction with a Hydrogeological study. The report of the findings, relating to the Hydrogeological study, will be issued by another party.

A preliminary geotechnical investigation was carried out at the Site in February, 2022 and a Preliminary Geotechnical Investigation Report No: 4515-22-GC were issued on March 4, 2022. Since only a limited number of boreholes had been carried out at the Site, during the preliminary investigation, additional boreholes were carried out to supplement the subsoil and groundwater conditions within the Site. The current geotechnical report has been revised based on the data obtained at the borehole logs during both investigations and an update on the proposed development, received from the client.

A review of the revised development plans indicated that the development of the Site will consist of four mid-rise buildings, identified as A, B, C and D, varying from 6 to 10 storeys in height, and a number of 3-storey townhouse blocks.

The geotechnical investigations were to determine the subsoil and groundwater conditions at the Site, affecting the design and construction of the conceptual structures. In particular, geotechnical data was to be provided for:

- General founding conditions
- Foundation design bearing pressures
- Construction recommendations
- Excavation recommendations

The report is provided on the basis of the above terms of reference and on an assumption that the design of structures will be in accordance with the applicable building codes and standards. If there are any changes in the design features relevant to the geotechnical analysis, our office should be consulted to review the design and to confirm the recommendations and comments provided in the report.

## 2.0 SITE CONDITION

The Site, approximately 6.45 hectares in area, is located between Watson Pkwy North and Watson Road North, beyond the end of Starwood Drive, in Guelph, Ontario.

At the time of the geotechnical investigation, the Site was a vacant parcel of land with sparse grass cover and scattered water accumulation. The site gradient within was relatively flat, slightly dropping towards the south and east.

### **3.0 INVESTIGATION PROCEDURE**

#### **Preliminary Investigation**

The field work for the preliminary investigation was carried out on February 17 and 18, 2022. A total of five sampled boreholes (22BH-1 to 22BH-5), extending to depths of 6.4m to 15.7m from grade, were carried out at the locations shown in Drawing No. 1.

#### **Additional Investigation**

The field work for the additional investigation was carried out on May 10, 11 and 12, 2022. A total of nine sampled boreholes (22BH-6 to 22BH-14), extending to depths of 4.7m to 15.7m from grade, were carried out at the locations shown in Drawing No. 1.

All the boreholes were advanced using a track mounted drill rig, equipped with continuous flight hollow stem augers, sampling rods and a drop hammer, supplied by a specialist drilling contractor. Soil samples were taken at 0.76m intervals to depths of 3.0m below the existing ground level. Below the depth, the sampling frequency was increased to 1.5m. The samples were obtained using a split spoon sampler in conjunction with Standard Penetration Tests using a driving energy of 475 joules (350 ft-lbs). The soil samples were identified and logged in the field and were carefully bagged for later visual identification and the determination of moisture content.

Groundwater observations were made in the boreholes during and upon the completion of drilling. Eight of the boreholes, 22BH-1, 22BH-3, 22BH-6 to 22BH-9, 22BH-12 and 22BH-13, were completed as monitoring wells to document the current static groundwater levels. The symbol (MW), besides the borehole identification, indicates a monitoring well. The groundwater records are presented in the borehole logs.

The locations of boreholes are shown on the appended Borehole Location Plan (Drawing No. 1). The ground elevations at the borehole locations were interpolated from the Topographic Survey of part of Lot 5, Concession 3, Division 'C', City of Guelph, prepared by Speight, Van Nostrand & Gibson Limited, dated February 2, 2004, provided to our office by the client.

## 4.0 SUMMARIZED SITE AND SUBSURFACE CONDITIONS

Reference is made to the appended Borehole Location Plan (Drawing No. 1), Logs of Boreholes (Drawing Nos. 2 to 17) and sections (Drawing Nos. 18 to 20), for details of field work, including soil classification, inferred stratigraphy, and groundwater observations carried out during and on completion of borehole drilling.

The boreholes revealed that the subsoil consisted of a layer of fill, overlying native deposits of sand and gravel, sandy silt till and silty sand deposits.

Brief descriptions of the subsoils, encountered at the borehole locations, were as follows:

### 4.1 Surface Course

It is our understanding that the Site was uplifted / graded with placement of fill in the past. Scattered vegetation was evident at the ground surface at the Site.

Topsoil, approximately 25mm to 125mm in thickness, was contacted at the ground surface at Boreholes 22BH-6 to 22BH-14 locations.

### 4.2 Fill

A layer of fill was contacted, at the ground surface at Boreholes 22BH-1 to 22BH-5 locations, and below the topsoil at Boreholes 22BH-6 to 22BH-14 locations. The fill consisted of a mixture of sandy silt to silty sand or sand, trace to some gravel with trace to some clayey silt and contained occasional minor rootlets and topsoil.

The fill, at all borehole locations, extended to depths of 0.6m to 3.5m from grade.

Based on the soil quality and the Standard Penetration N-values, in the range of 5 to 28 blows per 0.3m penetration, it appears that the fill might have been placed and compacted under some supervision. The N-values higher than 36 blows per 0.3m penetration, could be due to the presence of big gravel.

The in-situ moisture content of the soil samples obtained from the fill ranged from 8% to 23%, indicating moist to very moist conditions, with some wet pockets.

### 4.3 Sand and Gravel

Sand and gravel deposit was contacted at all borehole locations, below the fill, at depths of 0.6m to 3.5m from grade. The sand and gravel deposit contained some silty sand and / or sandy silt, with occasional cobbles. Some river sand and gravel was evident at Boreholes 22BH-2, 22BH-3, 22BH-10 and 22BH-12 locations.

Boreholes 22BH-3, 22BH-12 and 22BH-14 were terminated in the sand and gravel deposit at a depth of 6.6m, 5.6m and 4.7m from grade, respectively. The sand and gravel, at the remaining boreholes, extended to depths of 4.0m to 7.8m from grade.

Based on the Standard Penetration N-values of 16 to more than 100 blows per 0.3m penetration, the relative density of the sand and gravel deposit were compact to very dense, generally dense to very dense.

The in-situ moisture content of the soil samples retrieved from this deposit ranged from 2% to 25%, indicating moist to wet conditions.

Grain size analyses were carried out on two soil samples from this deposit, obtained from Boreholes 22BH-4 (SS6 – at a depth of 4.6m) and 22BH-12 (SS5 – at a depth of 3.0m), using mechanical sieves. The grain size distributions are shown on the appended Figure No. 1.

### 4.4 Sandy Silt Till

A sandy silt till deposit was contacted at Boreholes 22BH-1, 22BH-2, 22BH-4 to 22BH-11 and 22BH-13 locations, below the sand and gravel deposit, at depths of 4.0m to 7.8m from grade. The deposit consisted of a heterogeneous mixture of silt, sand, some clay, some gravel, seams of fine sand, with occasional layers of silty sand, sandy silt or clayey silt till.

Boreholes 22BH-2, 22BH-4 to 22BH-11 and 22BH-13 were terminated in the sandy silt till deposit at depths of 6.4m to 15.7m from grade. The sandy silt till deposit at Borehole 22BH-1 extended to a depth of 10.1m from grade.

A lower sandy silt till deposit was contacted at Borehole 22BH-1 location, below a silty sand deposit, at a depth of 12.5m from grade. Borehole 22BH-1 was terminated in the lower sandy silt till deposit at a depth of 15.7m from grade.

Based on the Standard Penetration N-values of 12 to more than 100 blows per 0.3m penetration, the relative density of the sandy silt till deposit was compact to very dense, generally dense to very dense.

The in-situ moisture content of the soil samples retrieved from these deposits ranged from 8% to 17%, indicating moist to very moist conditions, with some wet pockets.

A grain size analysis was carried out on one soil sample from this deposit, obtained from Borehole 22BH-8 (SS11 – at a depth of 12.2m), using both mechanical sieves and hydrometer methods. The grain size distribution is shown on the appended Figure No. 1.

#### **4.5 Silty Sand**

A silty sand deposit was contacted at Boreholes 22BH-1 location, below the sandy silt till deposit, at a depth of 10.1m from grade. The deposit was fine to medium grained and contained trace gravel.

The silty sand deposit, at Borehole 22BH-1 location, extended to a depth of 12.5m from grade.

Based on the Standard Penetration N-values of 79 to 83 blows per 0.3m penetration, the relative density of the silty sand deposit was very dense.

The in-situ moisture content of the soil samples retrieved from this deposit ranged from 16% to 18%, indicating very moist to wet conditions.

#### **4.6 Groundwater**

Free water was recorded in the open boreholes, 22BH-2, 22BH-4 to 22BH-13, at depths of 2.1m to 6.1m from grade, with wet cave-in at depths of 2.9m to 14.0m from grade, during and upon completion of drilling and sampling. Free water and wet cave-in could not accurately recorded at Boreholes 22BH-1 and 22BH-3 locations, and these boreholes were completed as monitoring wells to determine the static groundwater levels across the Site.

On March 1, 2022, the water levels, measured in the monitoring wells at Boreholes 22BH-1 and 22BH-3, were documented at depths of 3.56m and 2.16m from grade, respectively. On May 26, 2022, the water levels, measured in the monitoring wells at Boreholes 22BH-1, 22BH-3, 22BH-6 to 22BH-9, 22BH-12 and 22BH-13, were documented at depths of 2.09m to 5.26m from grade.

The water levels, in the monitoring wells at Boreholes 22BH-1, 22BH-6 to 22BH-9, within the Parcel 1 area of the mid-rise buildings, were at depths of 2.92m to 5.26m from grade. The water levels, in the monitoring wells at Boreholes 22BH-3, 22BH-12 and 22BH-13, within the Parcel 2 area of townhouse blocks, were at depths of 2.09m to 4.07m from grade.

The documented water levels are listed below:

BH/WELL ID	Parcel	Ground Elevation	Depth of Well	Water Level at Depth / Elevation	
				March 1, 2022	May 26, 2022
22BH-1 (MW)	1	327.20m	15.2m	3.56m	3.69m / 323.51m
22BH-6 (MW)	1	328.20m	15.2m	-	5.26m / 322.94m
22BH-7 (MW)	1	327.80m	15.2m	-	4.12m / 323.68m
22BH-7S (MW)		327.80m	6.1m	-	4.07m / 324.84m
22BH-8 (MW)	1	326.75m	15.2m	-	2.92m / 323.83m
22BH-9 (MW)	1	326.48m	6.1m	-	2.96m / 322.76m
22BH-3 (MW)	2	325.26m	6.1m	2.16m	2.09m / 323.17m
22BH-12 (MW)	2	325.55m	5.8m	-	2.48m / 323.07m
22BH-12D (MW)		325.55m	10.7m	-	2.59m / 322.96m
22BH-13 (MW)	2	327.30m	6.1m	-	3.72m / 323.23m

Based on the moisture content profile of the soil samples retrieved from the boreholes, our field observations at the Site and the water levels measured in monitoring wells, it is our opinion that the depths of the free water represent a continuous groundwater table within sand and gravel, and silty sand deposits.



## 5.0 RECOMMENDATIONS

A review of a set of Architectural Drawings for Feasibility Study, prepared by Turner Fleischer, dated April 20, 2022, indicated that the development of the Site, Parcel 1 + Parcel 2, will consist of four 6 to 10-storey buildings with one level of underground parking in Parcel 1 area (along Watson Parkway North), and a number of townhouse blocks, without basements, in Parcel 2 area, with the associated roadways or parking lot. The ground floor elevations of the proposed buildings and townhouses were not known at the time of preparation of this report. We have assumed that the finished ground floor elevations will be at or above the existing ground level and the slab-on-grade of P1 level will be at depths of 3.0m from grade. The founding levels of the spread footings are assumed to be 1.0m lower than the above slab-on-grade depths, i.e. at or below depths of 4.0m from grade. However, the elevator and the surrounding foundations are anticipated to be deeper than the above assumed levels, at depths of 6.0m from grade.

These assumed foundation depths of the 8 to 10-storey buildings are approximately 1.1m to 3.1m below the current static groundwater level at Boreholes 22BH-1 and 22BH-6 to 22BH-9 locations, documented at the monitoring wells. Unless a permanent groundwater control system is used to maintain the water level a minimum of 0.5m below the proposed slab-on-grade elevations, we recommend that the part of the underground parking, below the highest anticipated water level, should be designed as a water tight structure and consideration should, therefore, be given to use a raft slab as the foundation of the proposed structure. Based on the borehole profiles, our comments and recommendations are as follows:

### 5.1 Site Preparation

During the site preparation, the contractor must allow for removal of topsoil, deleterious fill and material with high moisture and/or organic content, if encountered, within the building/townhouse envelopes, the access roads and parking area.

The existing fill, as revealed in the borehole locations, appears to have been compacted under some supervision and may be left in place in its current state, for the design and construction of the slab-on-grade of the proposed townhouse. To achieve uniform subgrade conditions, we recommend that after removal of any unsuitable surface soil, the subgrade should be proofrolled, after it has been reviewed by a soils engineer from our office.

Any new fill, placed within the proposed building areas, should consist of organics free soil and compacted in 200mm lifts to at least 98% of its Standard Proctor maximum dry density.

## 5.2 Foundation Design

The proposed buildings and townhouses can be supported on conventional spread/strip footings, founded on the native undisturbed strata of sand and gravel, and sandy silt till deposits.

The soils at the proposed building locations consist of predominately non-cohesive fill and native deposits, and vertical excavation, without side supports, will not be stable. Trench and pour method will not be feasible.

### **Parcel 2 - Townhouses (Boreholes 22BH-2 to 22BH-5 & 22BH-10 to 22BH-14)**

All footings should be founded in the native sand and gravel deposit. In some locations, the footings will have to be taken through the fill and founded in the underlying native sand and gravel deposit. Spread / strip footings founded on the native sand and gravel deposit can be designed using the following bearing pressures:

- at Factored Ultimate Limit State = 220 kPa
- at Serviceability Limit State = 150 kPa

All perimeter footings should be founded a minimum of 1.2m below the final outside grade.

### **Parcel 1 – 8 to 10-Storey Buildings (Boreholes 22BH-1 & 22BH-6 to 22BH-9)**

The ground floor elevations of the proposed buildings were not known at the time of preparation of this report. We have assumed that slab-on-grade of P1 level will be at depths of 3.0m from grade. The founding levels of the spread footings are assumed to be 1.0m lower than the above slab-on-grade depths, i.e. at or below depths of 4.0m from grade. However, the elevator and the surrounding foundations are anticipated to be deeper than the above assumed levels, at depths of 6.0m from grade.

The subsoil at the assumed founding depths of 4.0m to 6.0m from grade are anticipated to consist of compact to very dense sand and gravel to sandy silt till deposits at Boreholes 22BH-1 and 22BH-6 to 22BH-9 locations.

Spread or strip footings, founded in the compact to very dense sand and gravel to sandy silt till deposits, at depths of 4.0m to 6.0m from grade, at Boreholes 22BH-1 and 22BH-6 to 22BH-9 locations, can be designed using the following bearing pressures:

- at Factored Ultimate Limit State = 450 kPa
- at Serviceability Limit State = 300 kPa

The native soils below the slab-on-grade of the buildings, at an assumed depth of 3m below grade, consists primarily of very permeable sand and gravel deposit. If the slab-on-grade is up to 1m below the highest recorded water level, the groundwater in this deposit can be maintained below the slab-on-grade by the installation of a network of sub-floor drainage system. The elevator shafts on the other hand will be significantly below the groundwater table and we recommend that the elevator shafts should be designed as water tight structures. It is, therefore very important that *Toronto Inspection Ltd.* should review these recommendations once the slab-on-grade elevations have been finalised.

If the proposed slab-on-grade depths are deeper than assumed and the groundwater table cannot be maintained below the slab-on-grade, each building will have to be designed as a watertight structure, below the highest anticipated static groundwater level, founded on a raft slab. The raft foundation can be designed using bearing pressures of 300 kPa at the Serviceability Limit State. A modulus of subgrade reaction of 40 MN/m<sup>3</sup> can be used for the design of raft slab on the compact to very dense deposits. The highest anticipated groundwater level should be as established by a hydrogeological study.

For the construction of the raft foundation, particularly at the elevator and the surrounding foundations, provision will have to be made to provide a space between the top of the raft and the slab-on-grade, for the installation of sewers and any other in-ground services.

The base of the raft foundation is anticipated to be up to 2.5m below the current static groundwater levels and will be subject to an uplift pressure of approximately 25 kPa. In addition, provision will have to be made for a rise in the groundwater levels, within the excavation, during heavy rain / wet season. We, therefore, recommend that the temporary dewatering system must not be decommissioned until the total combined weight of the raft and the structure is at least 33 kPa - a factor of safety of F=1.33. The structural engineer will have to certify the loads, before decommissioning the temporary dewatering system.

The total and differential settlement of footings, designed for the above Serviceability Limit State, will not exceed 25mm and 20mm, respectively.

All perimeter footings or any footings, which may be exposed to freezing conditions, should be placed below the frost penetration depth of 1.2m below the outside grade or provided with an equivalent thermal protection.

There is no official rule governing the footing depth for a fully enclosed unheated garage. For the one level of underground parking, the interior columns / walls and the perimeter wall footings can be founded at depths of 1.0m and 0.8m respectively below the top of the garage slab. However, footings adjacent to the fresh air ducts, the entrance of the garage and any other areas which may be exposed to the outside, a minimum frost cover of 1.2m should be provided. In addition, a nominal 50mm of Styrofoam insulation should be provided under the floor slab within the close proximity to the fresh air ducts.

It should be noted that the above recommendations for foundations have been analysed by *Toronto Inspection Ltd.* from the subsoil information obtained at the borehole locations. The bearing material, the interpretation between the boreholes and the recommendations of this report must be checked through field inspection provided by *Toronto Inspection Ltd.* to validate the information for use during the construction stage.

### **5.3 Floor Slab Construction**

The floor slab can be designed and constructed as a conventional slab-on-grade method. The subgrade should be thoroughly proof-rolled under the supervision of a geotechnical technician from *Toronto Inspection Ltd.* Any compressible, loose, or weak spots encountered during the proof rolling process, should be sub-excavated to a firm ground. Any backfill of the sub-excavated areas or new fill, below the slab-on-grade, should consist of organic free soils, compacted to at least 98% of its Standard Proctor maximum dry density (SPMDD).

A bedding consisting of at least 150 mm of granular A (OPSS Form 1010) or its approved equivalent, is recommended as a moisture barrier. The bedding should be compacted to at least 100% SPMDD. Based on the groundwater conditions encountered at Boreholes 22BH-1 and 22BH-6 to 22BH-9 locations, underfloor drainage systems, installed at a minimum depth of 0.6m on a positive gradient, at minimum of 6m centres, will be necessary below the slab-on-grade of P1 level to maintain the water level below the base of the slab.

For raft foundation design, the space between the top of the raft foundation and the slab-on-grade, for installation of sewers and other in-ground services, can be filled with 19mm clear stone. The floor slab can be poured directly over the clear stone backfill.

#### **5.4 Earthquake Consideration**

The Ontario Building Code requires that all buildings be designed to resist earthquake forces. In accordance with Table 4.1.8.4.A of the Ontario Building Code, the site classification for the Seismic Site Response is Class C (very dense soil).

The acceleration and velocity based site coefficients,  $F_a$  and  $F_v$ , should conform to Tables 4.1.8.4.B and 4.1.8.4.C. These values should be reviewed by the Structural Engineer.

#### **5.5 Excavation and Backfilling**

All excavations should comply with the Ontario Occupational Health and Safety Act. Any excavation in soil should be sloped back to a safe angle of 45° or flatter. For excavation into the saturated soils, the slope of excavation should be flattened to a safe condition.

Groundwater problem is not anticipated for excavations up to a depth of 2.1m to 2.9m from the existing ground level. Slight seepage may be encountered from the fill layer and/or from sand and gravel deposit. It is our opinion that the amount of water will not be great and can be handled by installing filtered sumps in the excavation during construction. The accumulated water can be removed by pumping. Below this depth, de-watering will be required in excavation for the foundations.

Selected on-site excavated soils can be reused for backfilling, provided they are free of organics and compressible material. The use of the compressible fill should be limited to backfilling of locations where future settlement will be of little consequence.

Topsoil and other compressible fill removed from the Site may be reused in landscape areas, subject to the approval of the landscape architect.

Bedding for the underground services, including catch basins and manholes, should consist of OPSS Granular A, 20mm crusher run limestone, or equivalent. Clear stone (HL-6 or 19mm maximum) may be used as bedding in saturated/wet subsoils provided that the stone bedding is completely surrounded by a geotextile, Terrafix 270R or equivalent.

At locations where adequate space will not be available for an open cut excavation, temporary shoring will have to be used to support the vertical faces of the excavation. The shoring design parameters and our recommendations on the installation and testing of the shoring system are provided in Appendix A of this report.

## 5.6 Lateral Earth Pressure

Where subsurface walls will retain unbalanced loads, the lateral earth pressure may be computed using the following equation:

$$P = K_o ( \gamma H + q )$$

where	P = Lateral earth pressure	kPa
	$K_o$ = Lateral earth pressure coefficient	0.4
	$\gamma$ = Bulk unit weight of the soil	21.5 kN/m <sup>3</sup>
	H = Depth of the wall below the finish grade	m
	q = Surcharge loads adjacent to the basement wall	kPa

The equation assumes that a permanent free draining system will be provided to prevent the buildup of hydrostatic pressure next to the wall.

For part of the structure is below the static groundwater table, it should be designed as a water tight structure. The lateral pressure of the structure, to a minimum of one metre above the static water level, should be computed using the following expression:

$$P_s = K ( \gamma' H_s + q ) + \gamma_w H_s$$

where	$P_s$ = Lateral earth pressure below the water table	kPa
	K = Lateral earth pressure coefficient	0.4
	$\gamma'$ = Submerged unit weight of the soil	11.7 kN / m <sup>3</sup>
	H = Depth of the wall below the water level	m
	$\gamma_w$ = Unit weight of water	9.8 kN / m <sup>3</sup>
	q = Surcharge loads adjacent to the basement wall	kPa

### 5.7 Permanent Perimeter Drainage

Permanent perimeter drains should be provided around the underground structure. At the shoring location, the permanent perimeter drain should consist of a prefabricated continuous blanket of Miradrain 6000 or its equivalent, as shown in Figure No. 2, provided permanent groundwater control is used to maintain the water level below the slab-on-grade. The installation of this type of vertical drainage system and its connections should be carried out as per the manufacturer’s specifications.

For an open cut excavation at the locations of basements, the typical backfill and drainage are shown in Figure No. 3.

### 5.8 Groundwater Control

Temporary lowering of the water will be required during construction stages, to at least 0.5m below the lowest footing / excavation elevation. The temporary dewatering system should not be decommissioned until the sub-floor drainage system is in place and the building construction has reached above the highest recorded groundwater level.

The hydrogeological study should be carried out and referred for source of the groundwater, the groundwater table and the temporary / permanent groundwater control.

### 5.9 Pavement Construction

The subgrade soils of the proposed roads, driveways and parking lot are anticipated to consist of sandy silt to silty sand with gravel and clayey silt.

The following minimum pavement design thicknesses are recommended based on the assumption that perforated sub-drains will be installed to prevent build-up of water in the granular bases of the pavement:

		<b>Light Duty</b>	<b>Heavy Duty</b>
		<b><u>Parking</u></b>	<b><u>Fire Routes</u></b>
<b>Asphaltic Concrete</b>	OPSS HL3 or equivalent	65mm	40mm
	OPSS HL8 or equivalent	-	60mm
<b>Base:</b>	OPSS Granular A or 20mm crusher-run	150mm	150mm
<b>Sub-base:</b>	OPSS Granular B or 50mm crusher-run	200mm	300mm

The above pavement thicknesses are based on the favourable site conditions and the construction being carried out during the drier time of the year and that the

subgrade is stable, not heaving under construction traffic. If the subgrade is wet and unstable, additional thickness of sub-base material may be required.

Roads and driveways to be assumed by the local municipality should be constructed to the municipal standards.

Following site grading, the subgrade of the entire pavement should be proof-rolled using a heavy vibratory roller. Any soft spots revealed by the proof-rolling should be sub-excavated and replaced with an approved dry material and compacted to at least 98% of its SPMDD. If the subgrade is wet and unstable, the wet material should be removed from the subgrade and additional thickness of subbase be used for road construction.

Continuous perforated, OPSS 405, longitudinal drains, minimum diameter of 100mm, should be used as sub-drains, on both sides of the roadways. The sub-drains should be installed on a positive gradient towards the outlets (collecting into catch basins), at a minimum depth of 800mm below the pavement level, to allow for a free flow of water. The backfill above the drains should comprise of free draining Granular B or its equivalent and should be continuous with the granular sub-base of the pavement. This will help in draining the pavement structure and minimize the differential heave of the pavement.

Catch basins and manholes should be backfilled with OPSS Granular B material. The catch basins should be perforated just above the drain level and the weep holes should be screened with a filtered fabric. This will help the pavement structure as well as alleviate the differential movement of the catch basins or the manholes due to the frost action.



## 6.0 GENERAL STATEMENT OF LIMITATION

The comments and recommendations presented in this report are based on the subsoil and ground water conditions encountered at the borehole locations, indicated in the borehole location plan, and are intended for the guidance of the design engineer. Although we consider this report to be representative of the subsurface conditions at the subject property, the soil and the ground water conditions between and beyond the borehole locations may differ from those encountered at the time of our investigation and may become apparent during construction. Any contractor bidding on, or undertaking the works, should decide on their own investigation and interpretations of the groundwater and the soil conditions between the borehole locations.

Any use and/or the interpretation of the data presented in this report, and any decisions made on it by the third party are responsibility of the third parties. The responsibility of *Toronto Inspection Ltd.* is limited to the accurate interpretation of the soil and ground water conditions prevailing in the locations investigated and accepts no responsibility for the loss of time and damages, if any, suffered by the third party as a result of decisions or actions based on this report.

Any legal actions arising directly or indirectly from this work and/or *Toronto Inspection Ltd.'s* performance of the services shall be filed no longer than two years from the date of *Toronto Inspection Ltd.'s* substantial completion of the services. *Toronto Inspection Ltd.* shall not be responsible to the client for lost revenues, loss of profits, cost of content, claims of customers, or other special indirect, consequential, or punitive damages.

To the fullest extent permitted by law, the client's maximum aggregate recovery against *Toronto Inspection Ltd.*, its directors, employees, sub-contractors, and representatives, for any and all claims by clients for all causes including, but not limited to, claims of breach of contract, breach of warranty and/or negligence, shall be the amount of the fee paid to *Toronto Inspection Ltd.* for its professional services rendered under the agreement with respect to the particular site which is the subject of the claim by the client.

**TORONTO INSPECTION LTD.**

**David S. Wang, P.Eng.**  
Senior Engineer



**Upkar S. Sappal, P.Eng.**  
Principal Engineer





Toronto Inspection Ltd.

**Drawings & Figures**

Borehole Location Plan

Borehole Logs

Section




Gradation Curve

Permanent Perimeter Drainage System

Suggested Backfill and Drainage System



**LEGEND:**

-  Borehole and Monitoring Well Location  
(February 2022 Drilling)
-  Borehole and Monitoring Well Location  
(May 2022 Drilling)
-  Site Boundary

NOT TO SCALE

**Toronto Inspection** INC.  
 GEO-ENVIRONMENTAL CONSULTANTS

110 Konrad Crescent,  
 Unit 16  
 Markham, Ontario  
 L3R 9X2

Tel: 905-940 8509      Fax: 905-940 8192      Email : TIL@torontoinspection.com

TITLE: Borehole and Monitoring Well Location Plan			
LOCATION: 115 Watson Parkway North (& Starwood Drive), Guelph, Ontario			
PROJECT NO.	4515-22-GC	DATE :	June 2022
		DRAWING NO.	1

Date Drilled: 2/17/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression

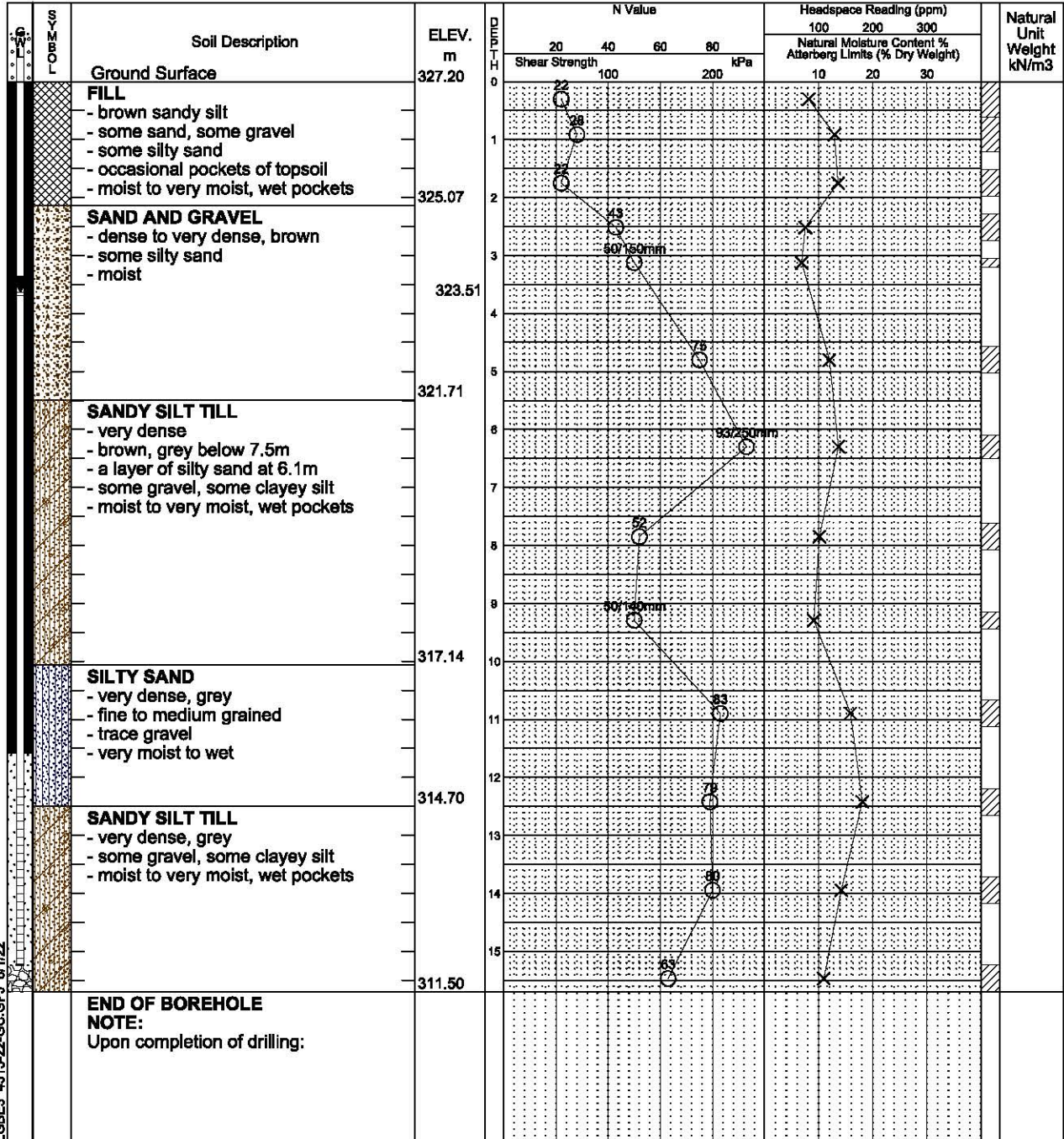


Datum: Geodetic

Field Vane Test



Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
Mar. 1, 2022	3.56m	
May 26, 2022	3.69m	

Date Drilled: 2/17/22

Auger Sample



Headspace Reading (ppm)



SPT (N) Value



Natural Moisture



Drill Type: Track Mounted Drill Rig

Dynamic Cone Test



Plastic and Liquid Limit



Datum: Geodetic

Shelby Tube



Unconfined Compression



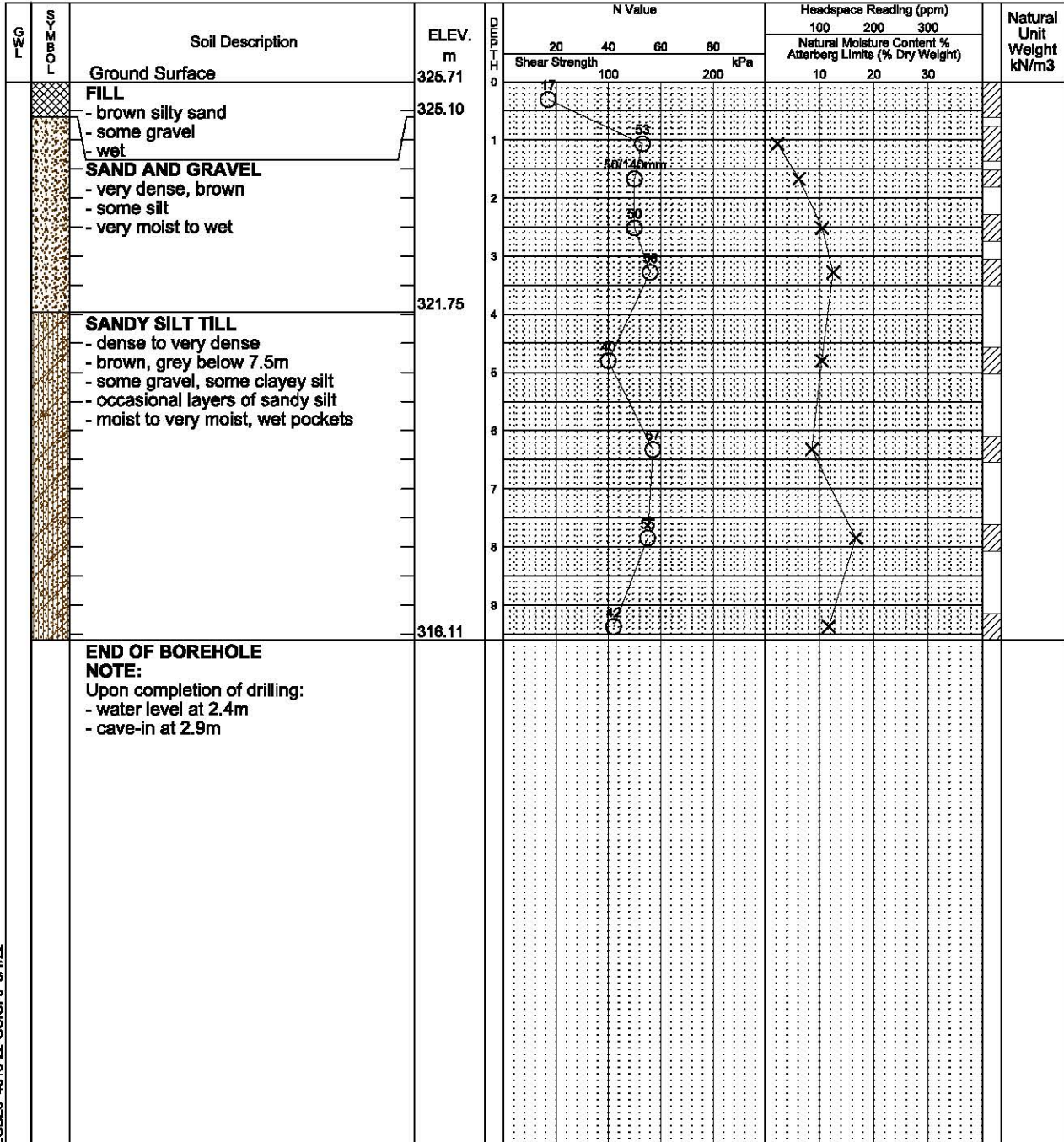
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Date Drilled: 2/18/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



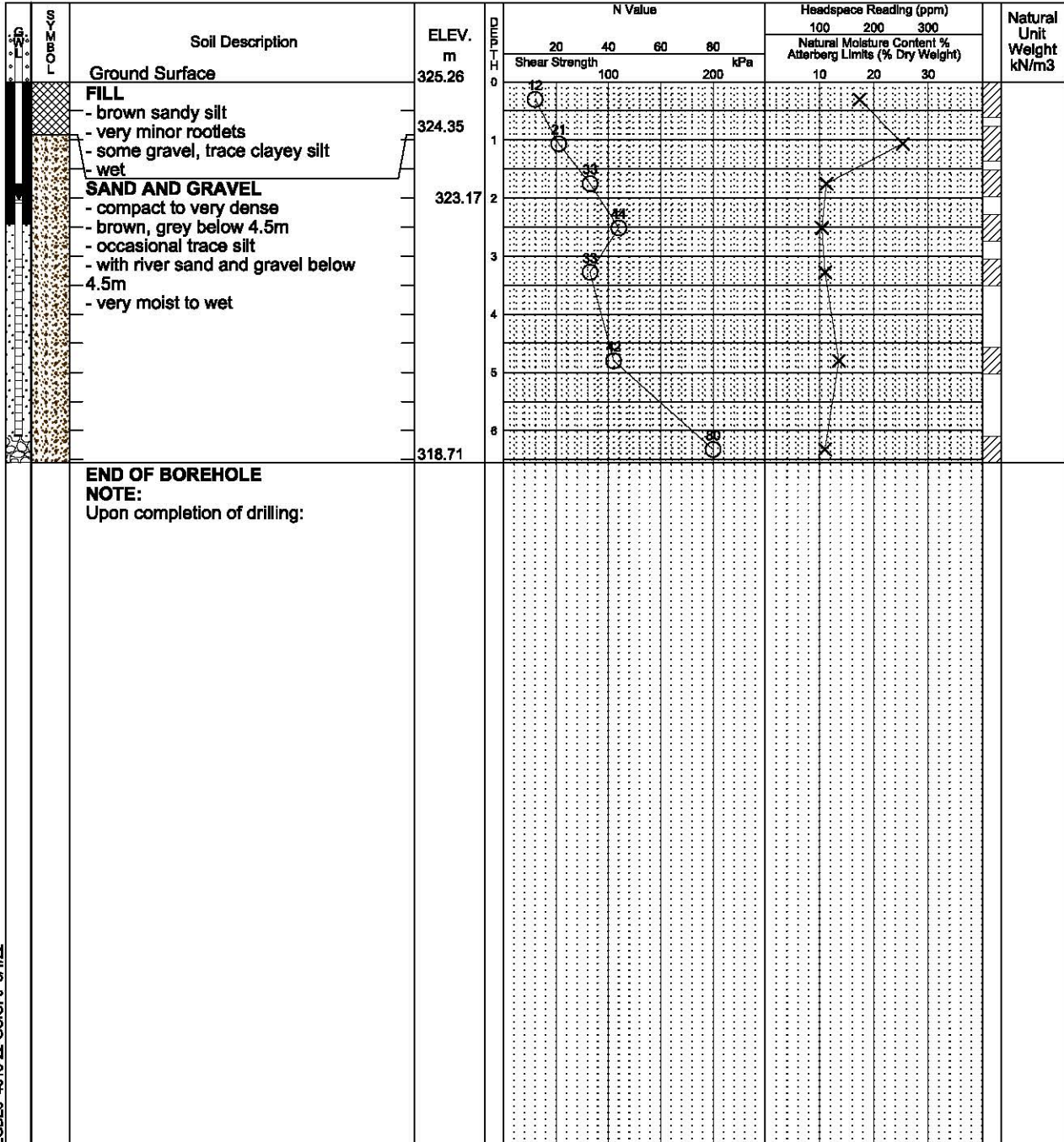
Field Vane Test



% Strain at Failure



Penetrometer



Date Drilled: 2/18/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



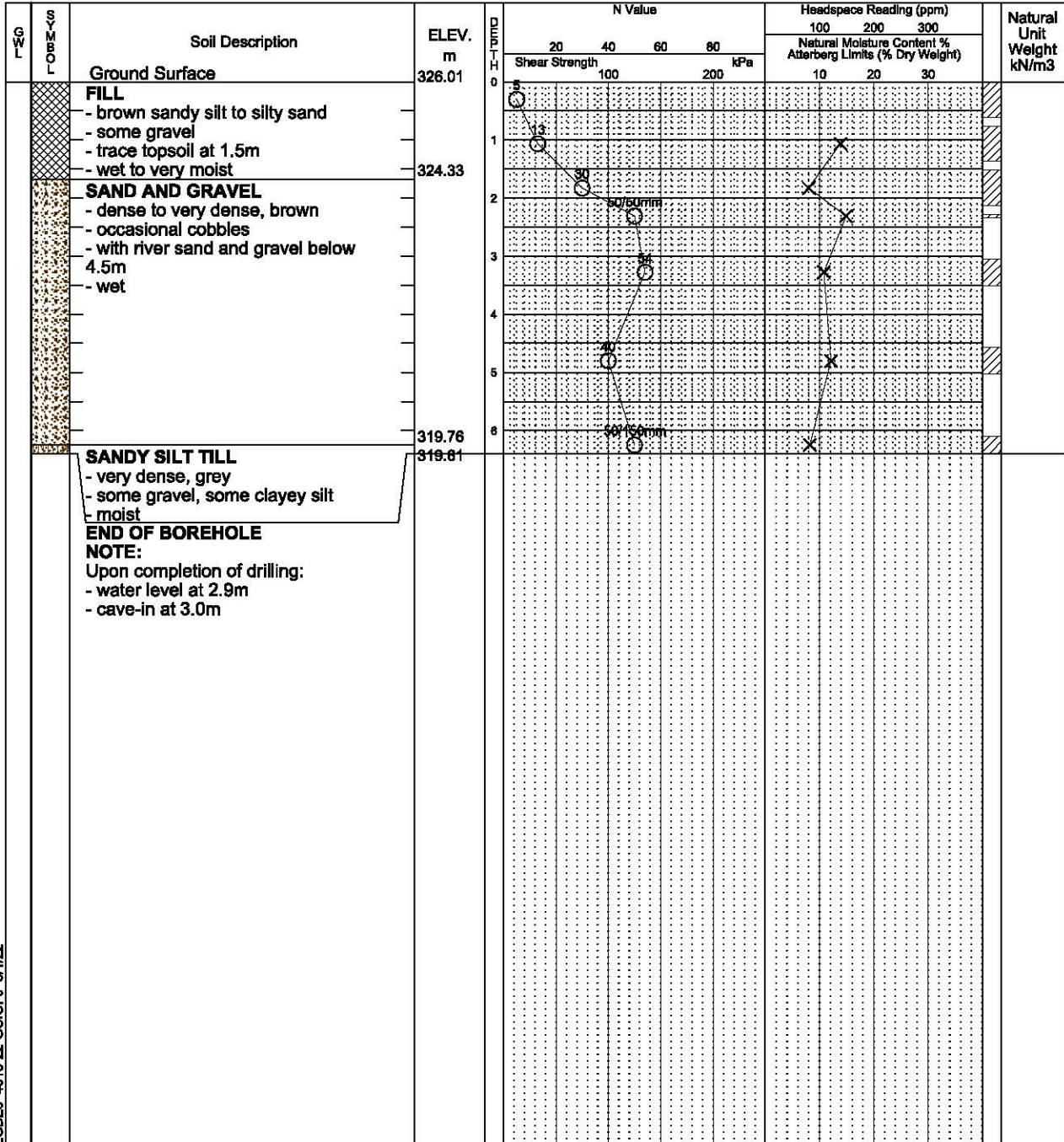
Field Vane Test



% Strain at Failure



Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

LGBE3 4515-22-GC.GPJ 8/1/22

Date Drilled: 2/17/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



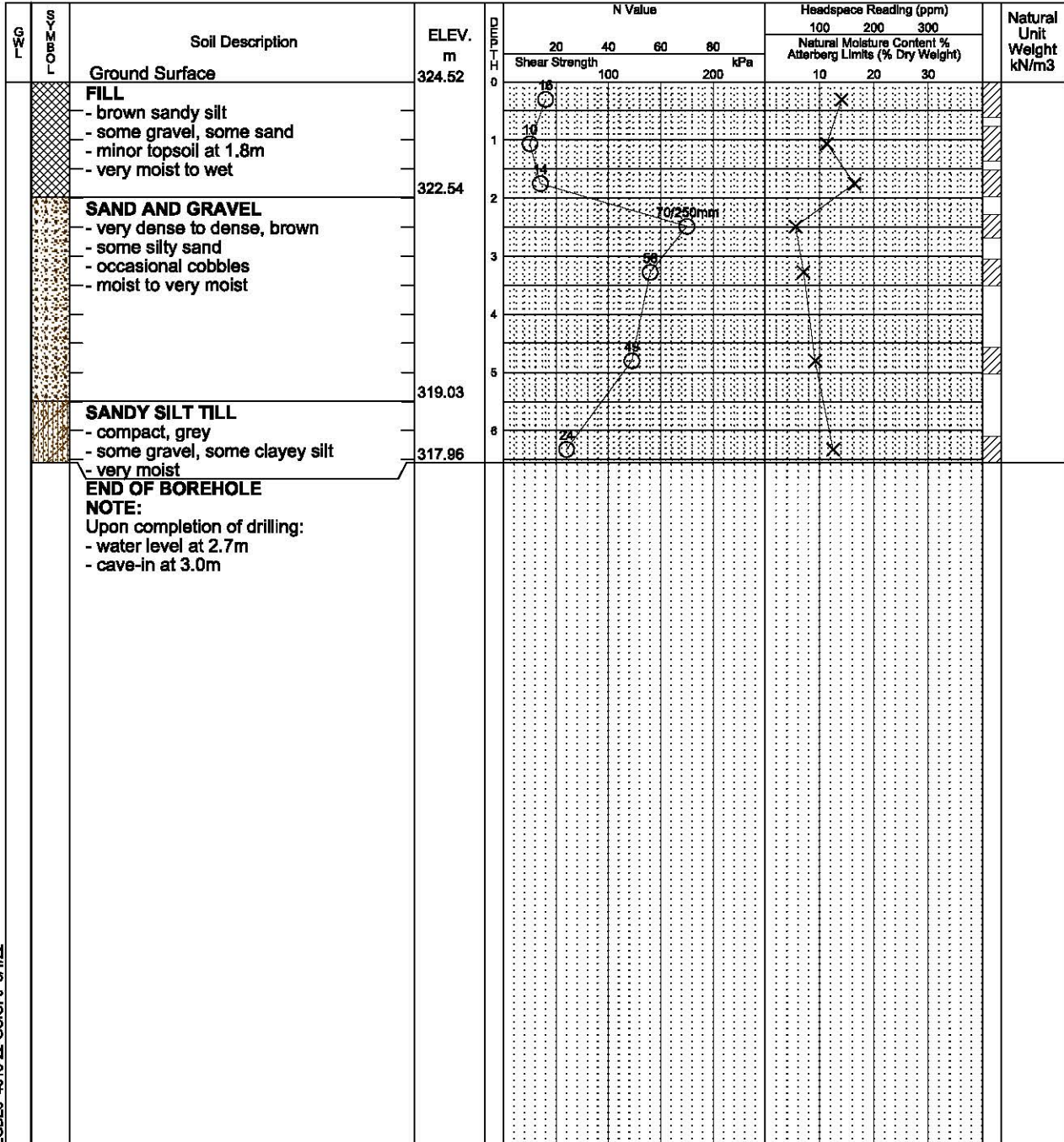
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)



Date Drilled: 5/11/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



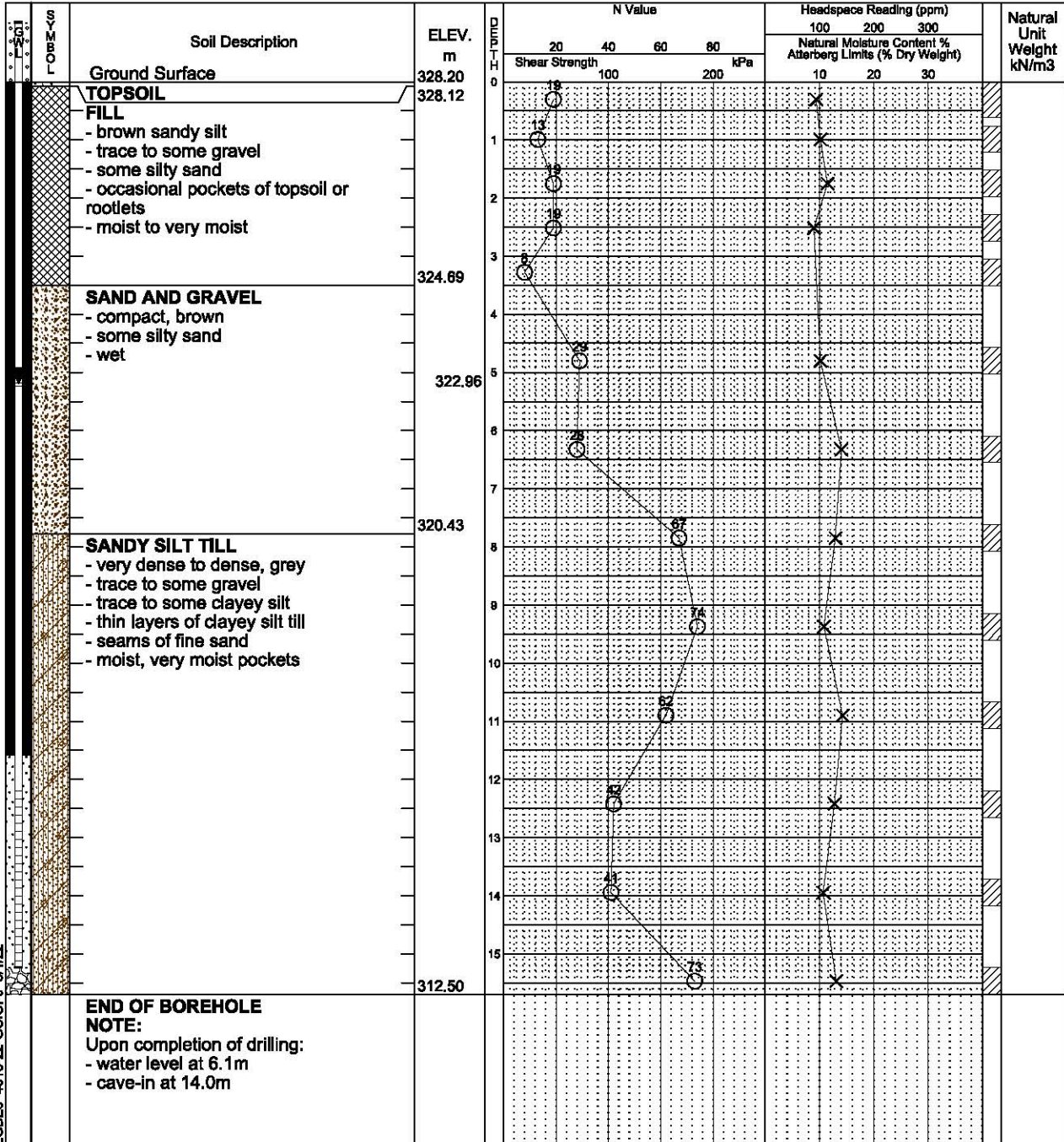
Field Vane Test



% Strain at Failure



Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	5.26m	

Date Drilled: 5/10/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



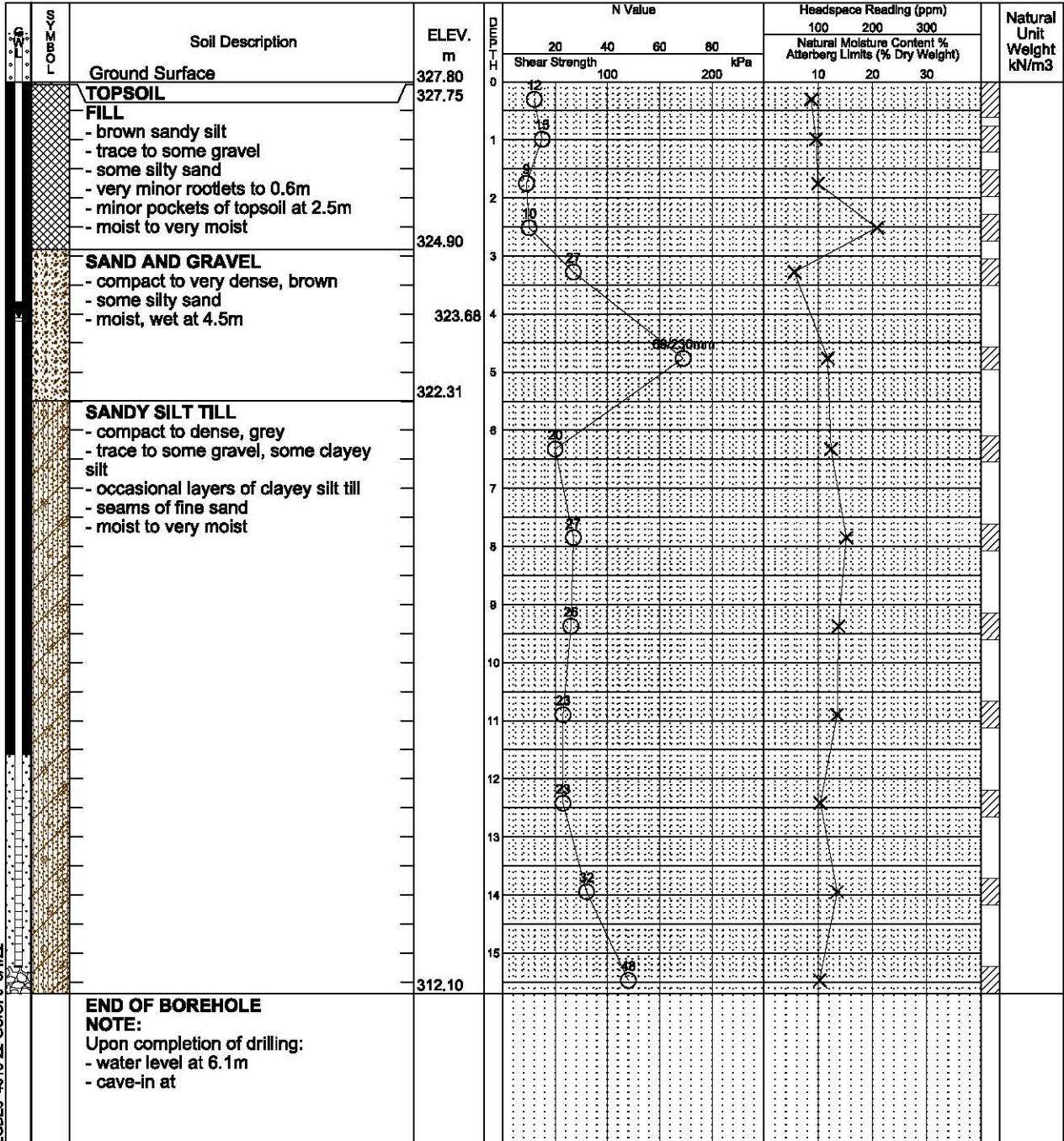
Field Vane Test



% Strain at Failure



Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	4.12m	

Project No. 4515-22-GC

# Log of Borehole 22BH-07S (MW)

Dwg No. 9

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 115 Watson Parkway North (& Starwood Drive), Guelph, Ontario

Date Drilled: 5/10/22

Auger Sample

Headspace Reading (ppm)

SPT (N) Value

Natural Moisture

Dynamic Cone Test

Plastic and Liquid Limit

Shelby Tube

Unconfined Compression

Drill Type: Track Mounted Drill Rig

Field Vane Test

% Strain at Failure

Datum: Geodetic

DEPTH (m)	Soil Description	ELEV. (m)	N Value				Headspace Reading (ppm)			Natural Unit Weight (kN/m <sup>3</sup> )
			20	40	60	80	100	200	300	
			Shear Strength (kPa)				Natural Moisture Content % Atterberg Limits (% Dry Weight)			
0	Ground Surface <b>NO SAMPLING</b> - straight drill to 6.12m	327.80								
1										
2										
3										
4		323.75								
5										
6	<b>END OF BOREHOLE</b> <b>NOTE:</b> Upon completion of drilling:	321.70								

LBBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	4.07m	

Date Drilled: 5/10/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



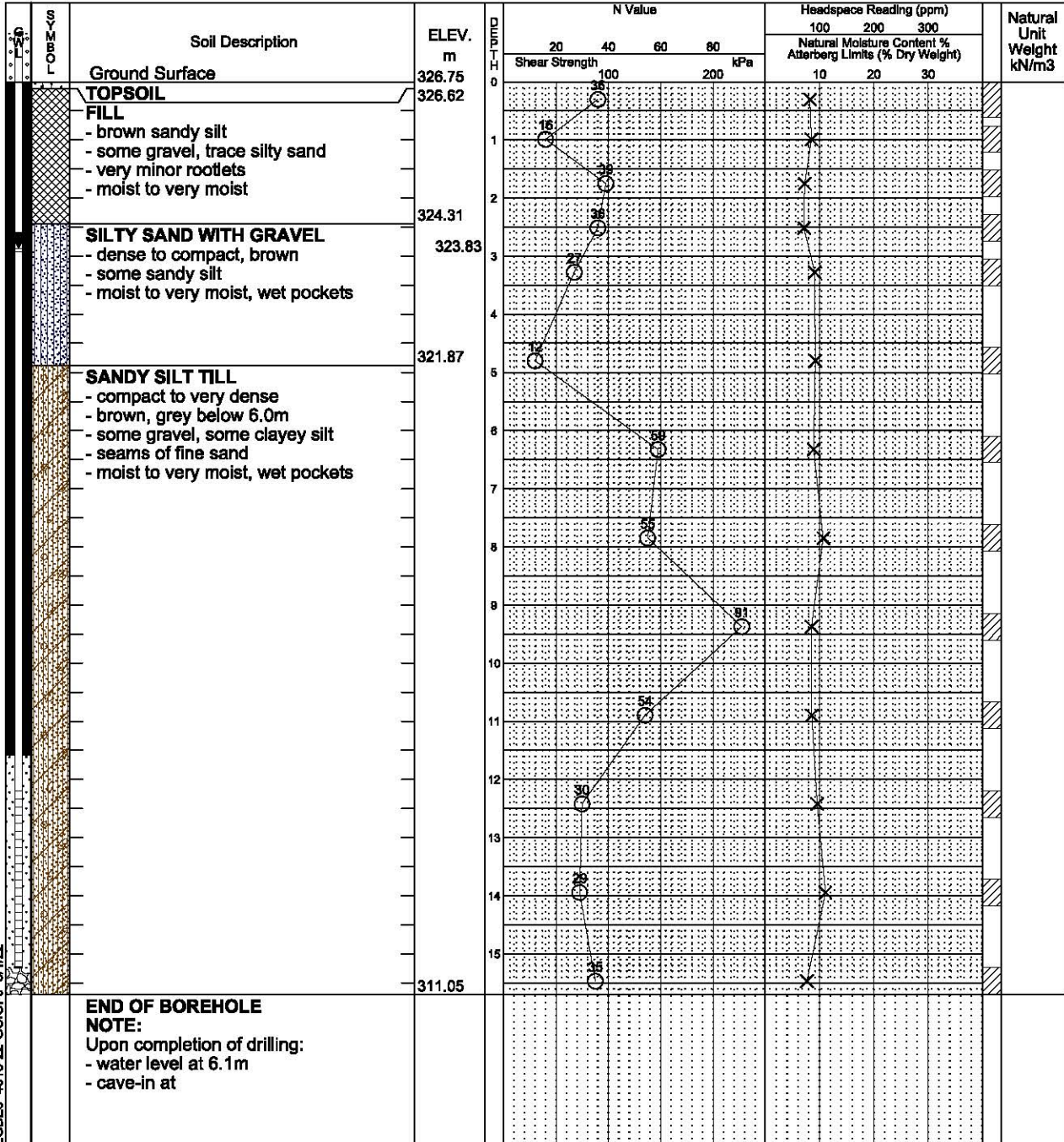
Field Vane Test



% Strain at Failure



Penetrometer



NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	2.92m	

Date Drilled: 5/12/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



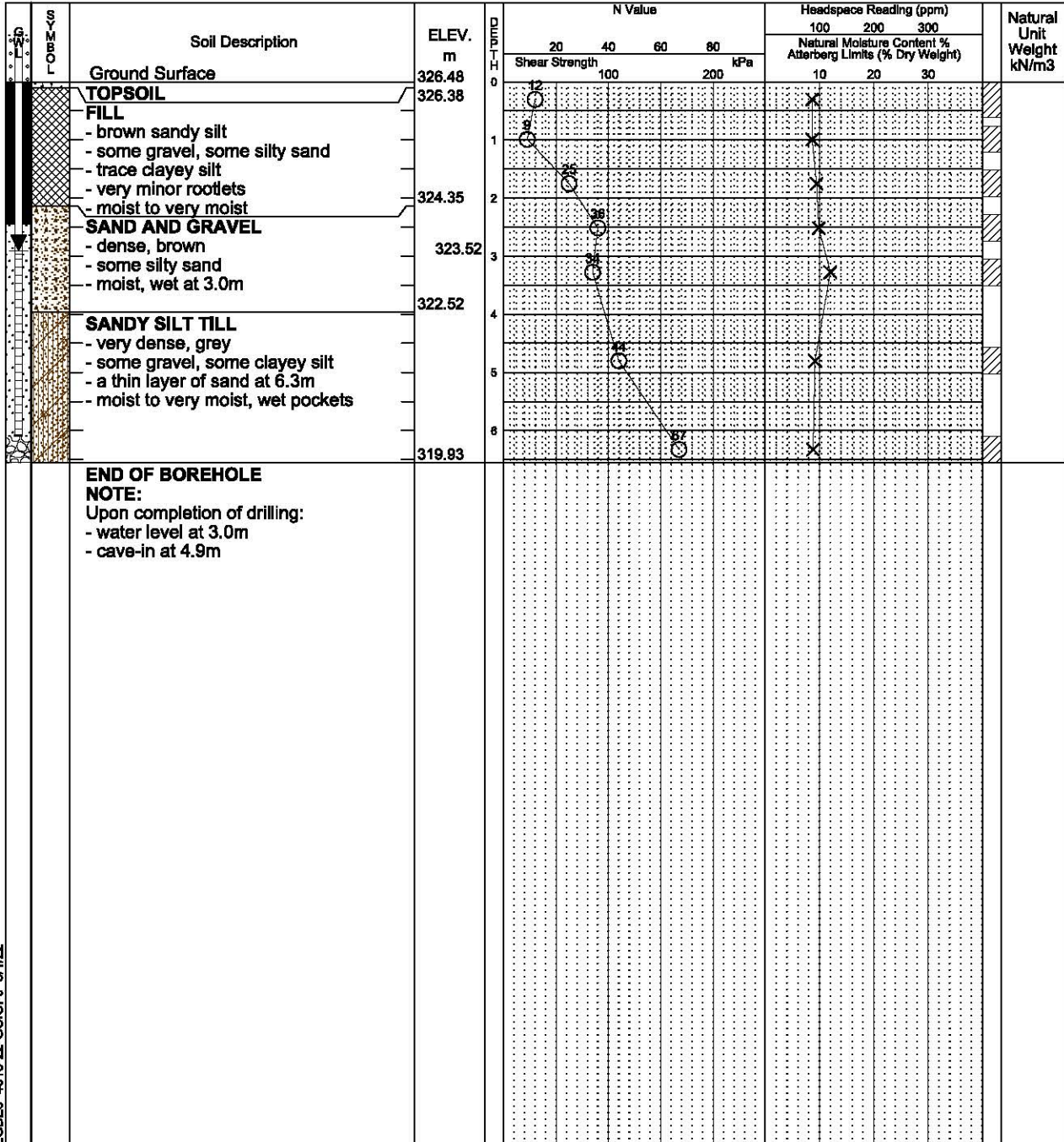
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	2.96m	

Date Drilled: 5/12/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



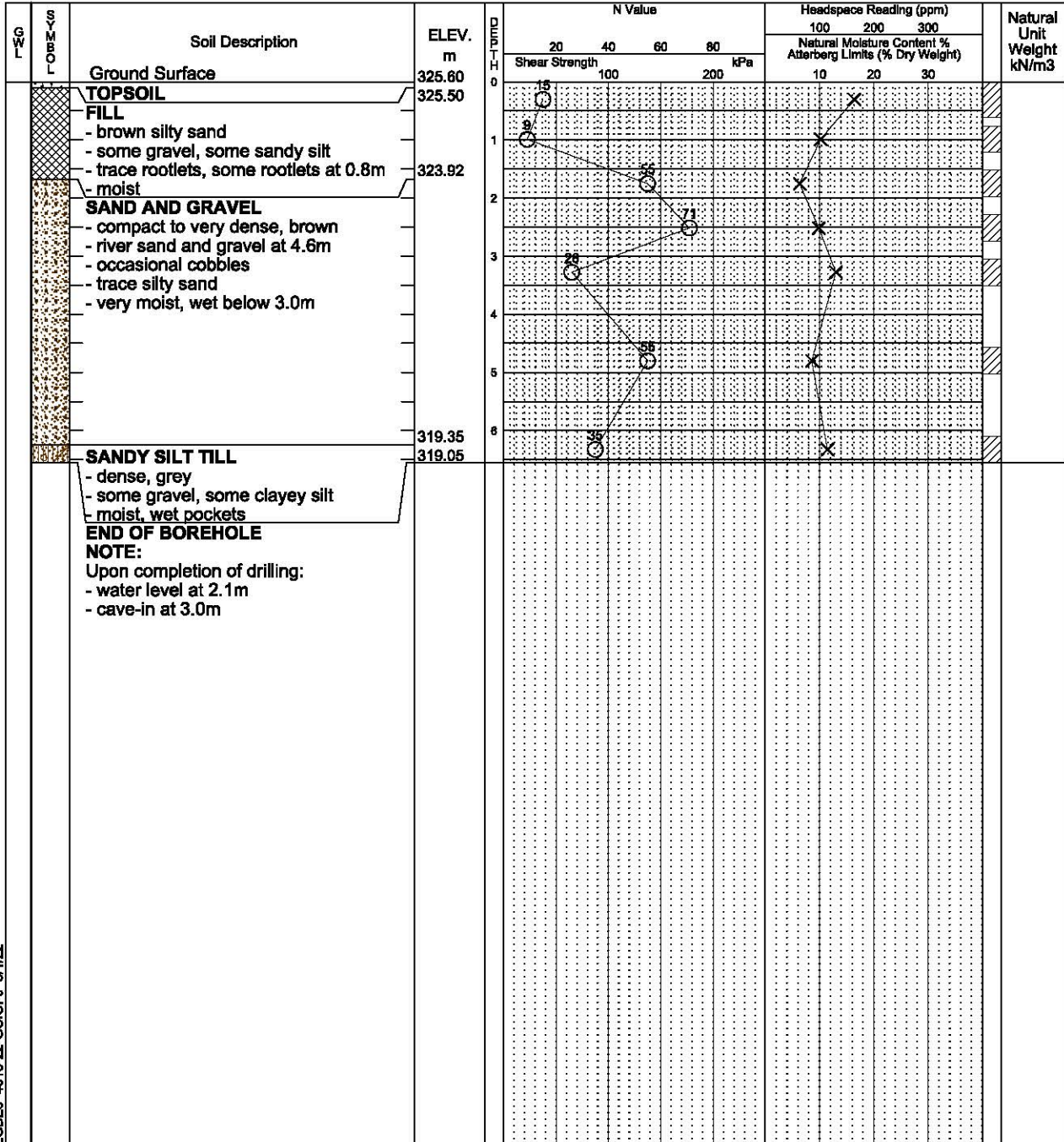
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Date Drilled: 5/11/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



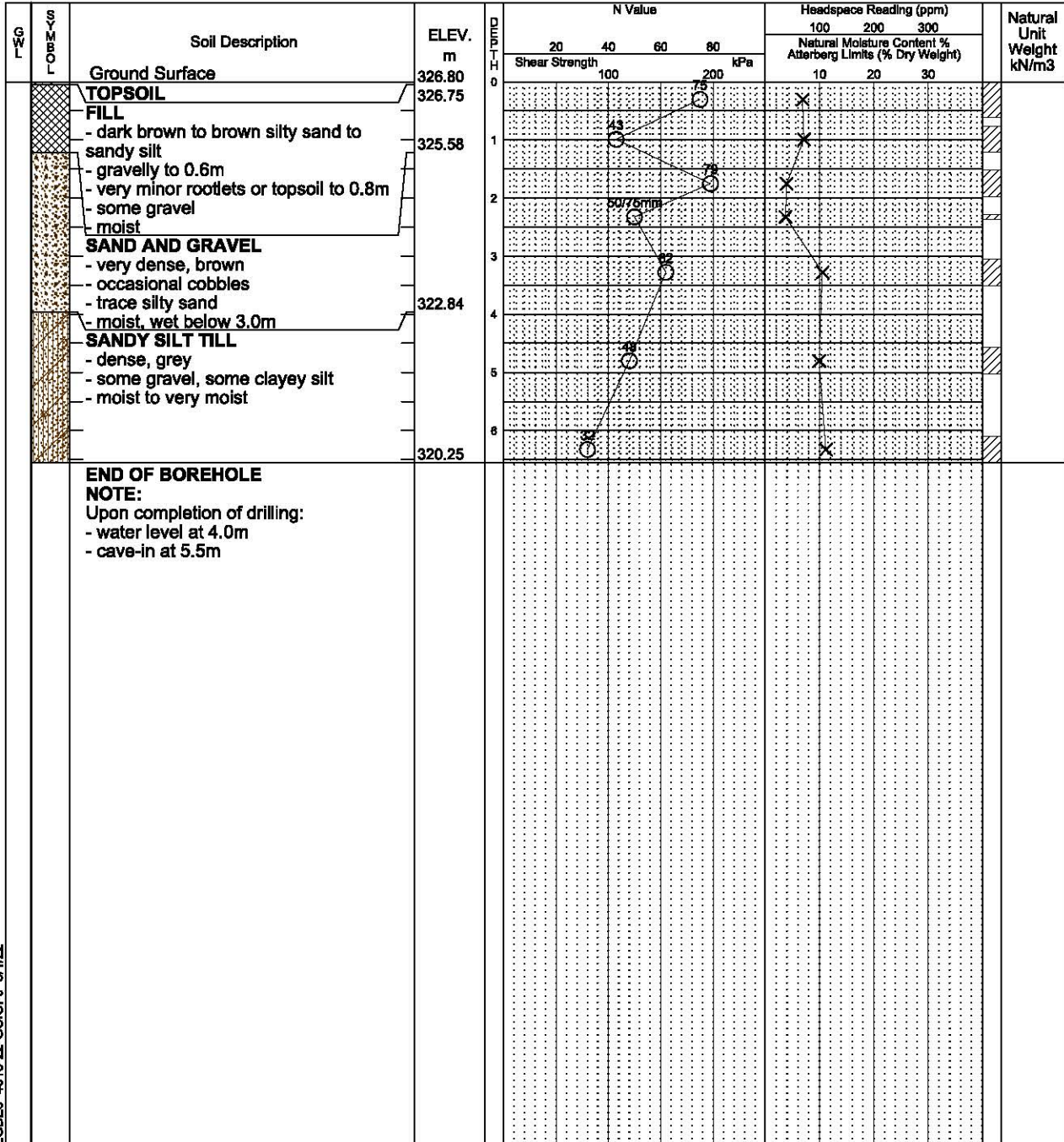
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)

Date Drilled: 5/12/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



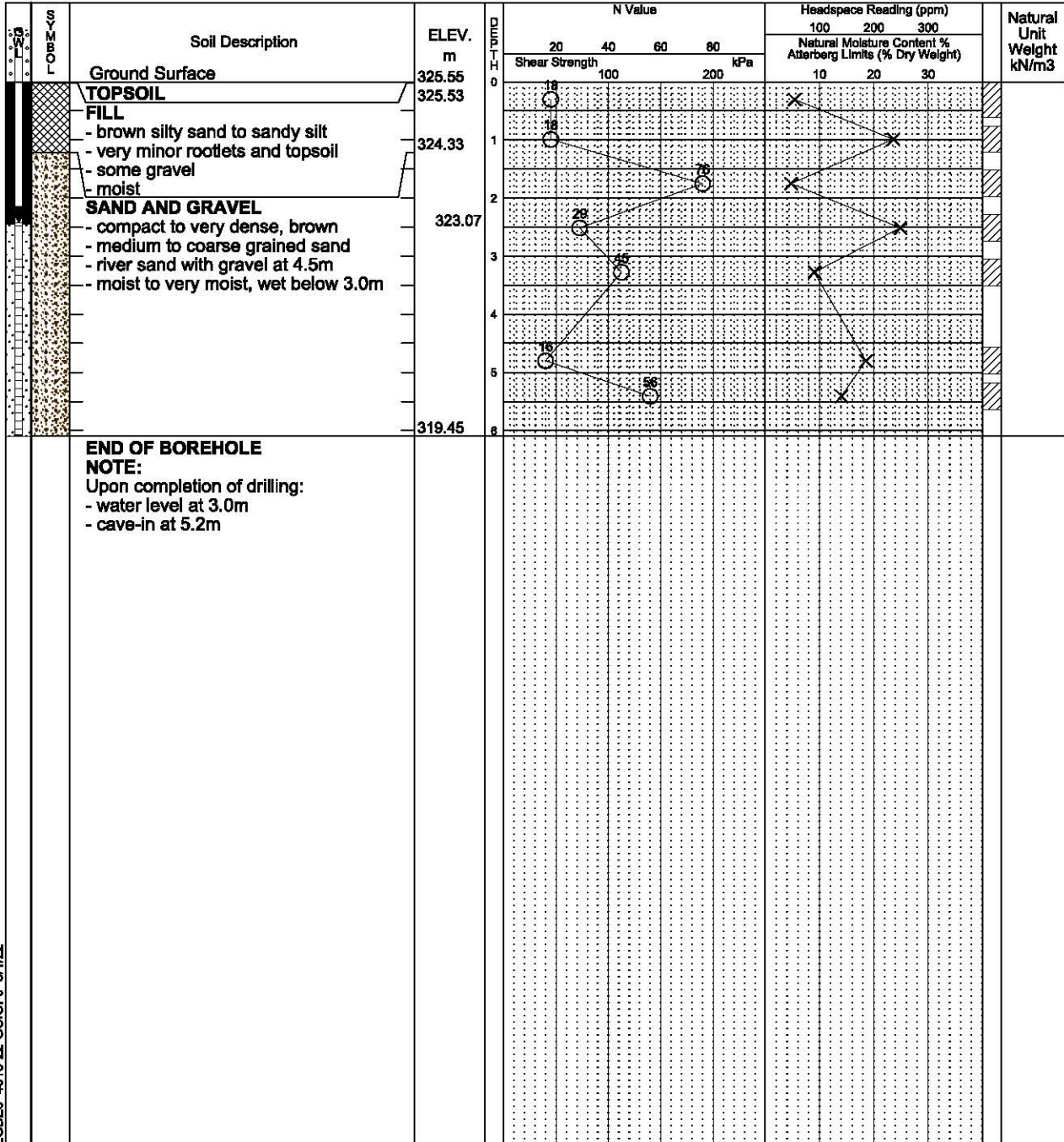
Field Vane Test



% Strain at Failure



Penetrometer



LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	2.48m	



Project No. 4515-22-GC

# Log of Borehole 22BH-12D (MW)

Dwg No. 15

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 115 Watson Parkway North (& Starwood Drive), Guelph, Ontario

Date Drilled: 5/12/22

Auger Sample



Headspace Reading (ppm)



SPT (N) Value



Natural Moisture



Drill Type: Track Mounted Drill Rig

Dynamic Cone Test



Plastic and Liquid Limit



Datum: Geodetic

Shelby Tube



Unconfined Compression



Field Vane Test



% Strain at Failure



Penetrometer



DEPTH m	Soil Description	ELEV. m	N Value				Headspace Reading (ppm)			Natural Unit Weight kN/m <sup>3</sup>
			Shear Strength kPa				100	200	300	
			20	40	60	80	Natural Moisture Content % Atterberg Limits (% Dry Weight)			
0	Ground Surface <b>NO SAMPLING</b> - straight drill to 9.9m	325.55								
1										
2		322.96								
3										
4										
5										
6										
7										
8										
9										
10		314.88								
	<b>END OF BOREHOLE</b> <b>NOTE:</b> Upon completion of drilling:									

LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	2.59m	

Date Drilled: 5/11/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



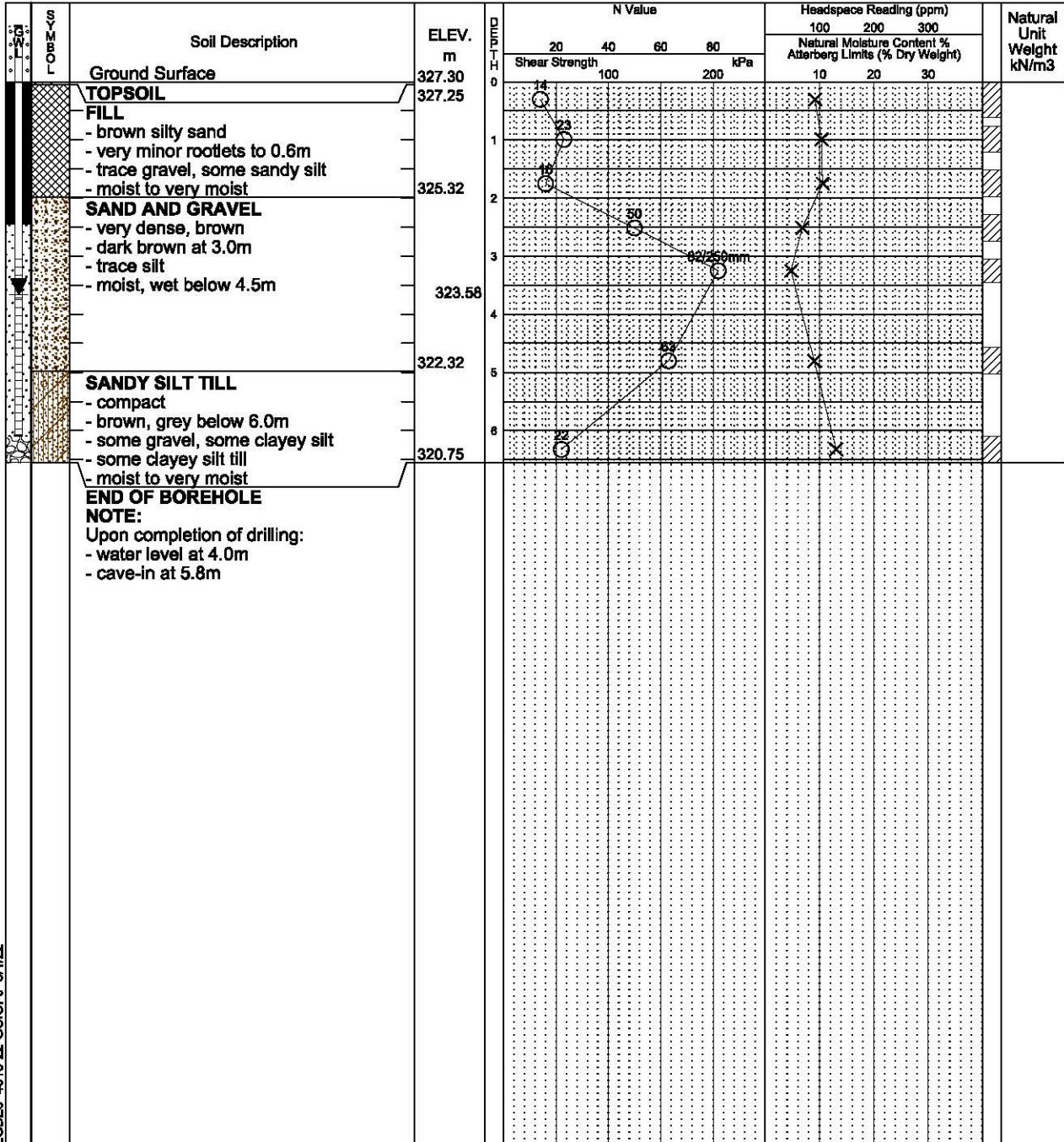
Field Vane Test



% Strain at Failure



Penetrometer



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Time	Water Level (m)	Depth to Cave (m)
May 26, 2022	3.72m	

LBBE3 4515-22-GC.GPJ 8/1/22

Date Drilled: 5/10/22

Auger Sample



Headspace Reading (ppm)



Drill Type: Track Mounted Drill Rig

SPT (N) Value



Natural Moisture



Datum: Geodetic

Dynamic Cone Test



Plastic and Liquid Limit



Shelby Tube



Unconfined Compression



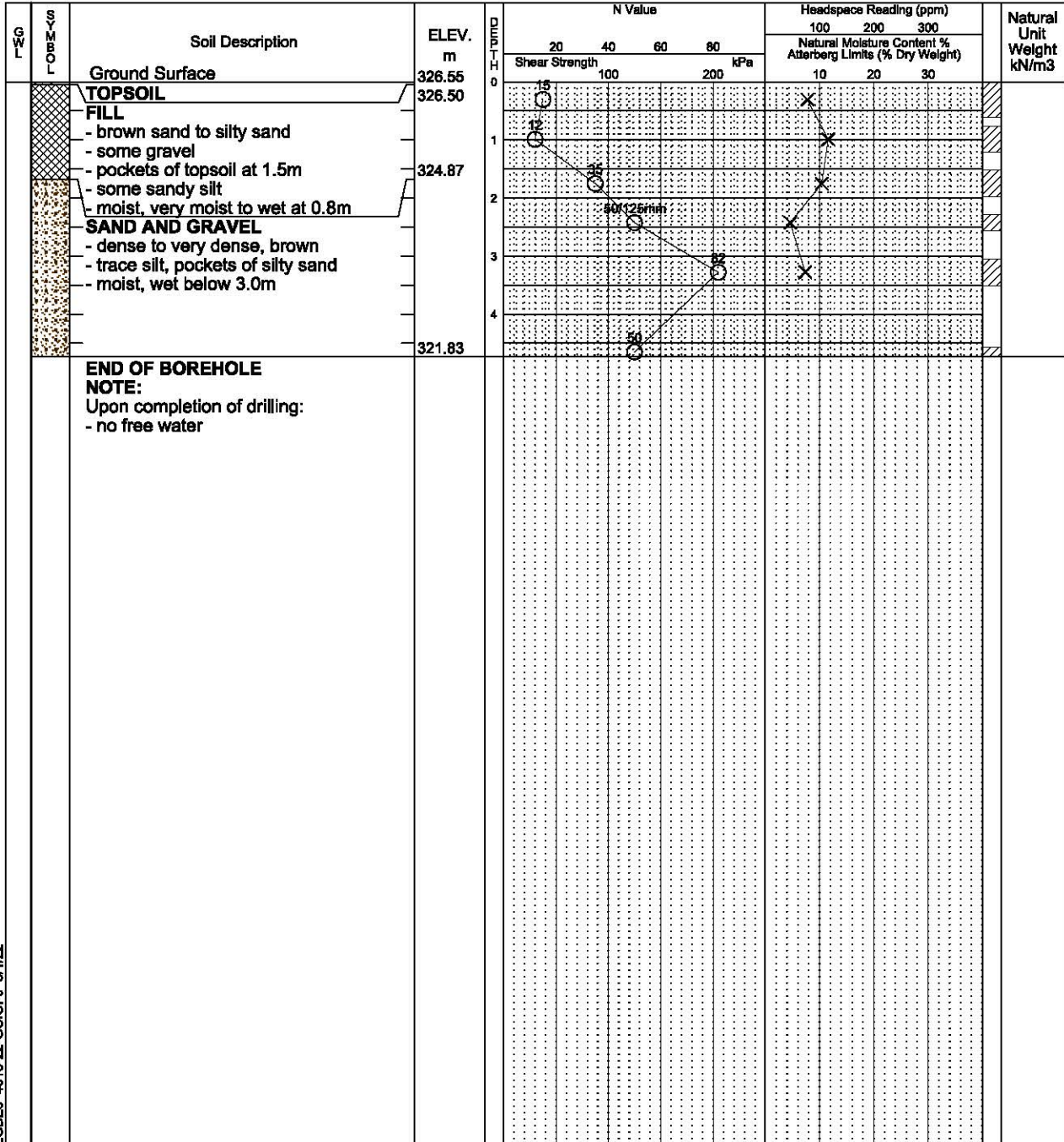
Field Vane Test



% Strain at Failure



Penetrometer

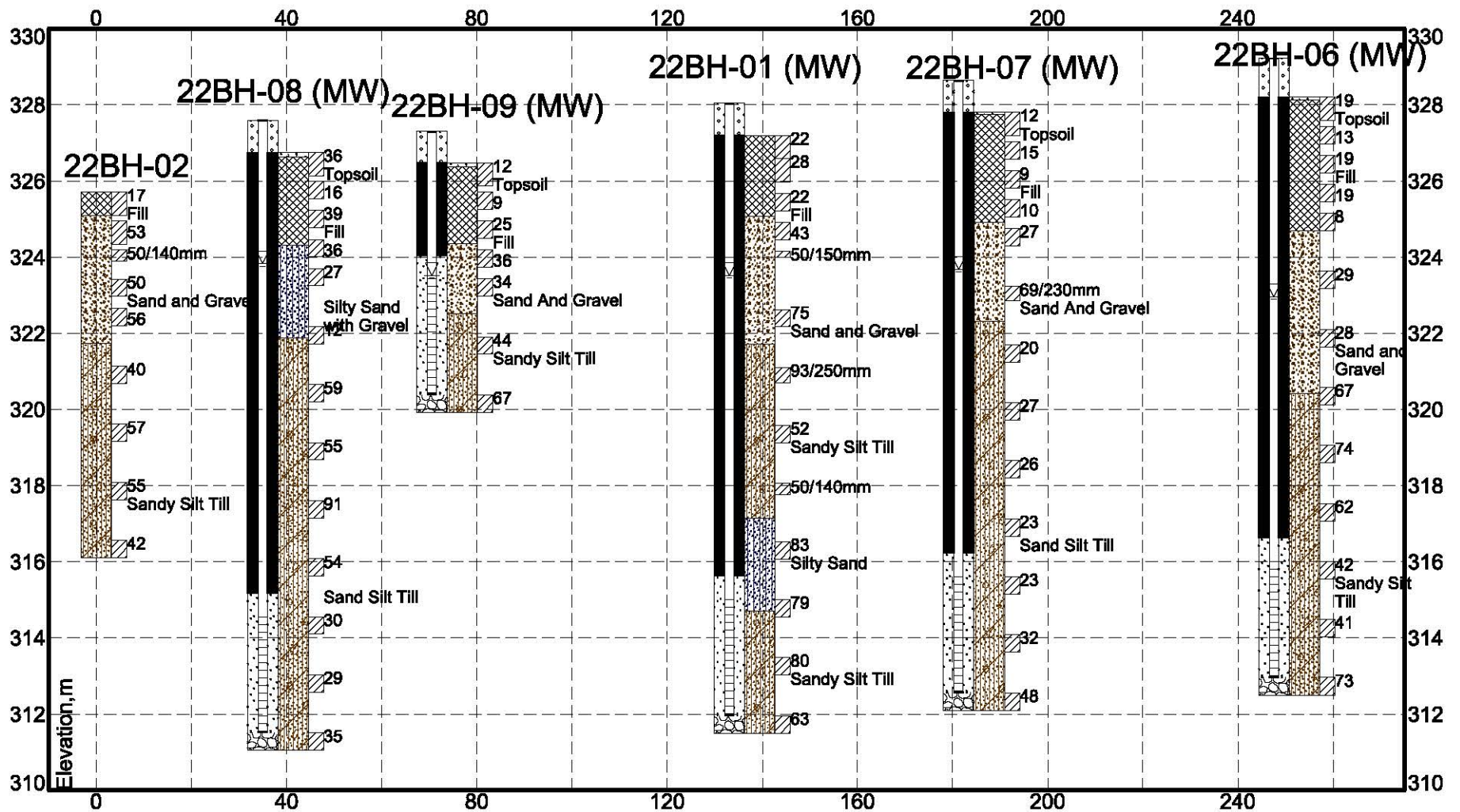


LGBE3 4515-22-GC.GPJ 8/1/22

NOTE: THE BOREHOLE DATA NEEDS INTERPRETATION ASSISTANCE BY TORONTO INSPECTION LTD. BEFORE USE BY OTHERS

## Toronto Inspection Ltd.

Time	Water Level (m)	Depth to Cave (m)



Borehole No	Elev.	Depth
22BH-01 (MW)	327.2	15.7
22BH-02	325.7	9.6
22BH-06 (MW)	328.2	15.7
22BH-07 (MW)	327.8	15.7
22BH-08 (MW)	326.7	15.7
22BH-09 (MW)	326.5	6.6

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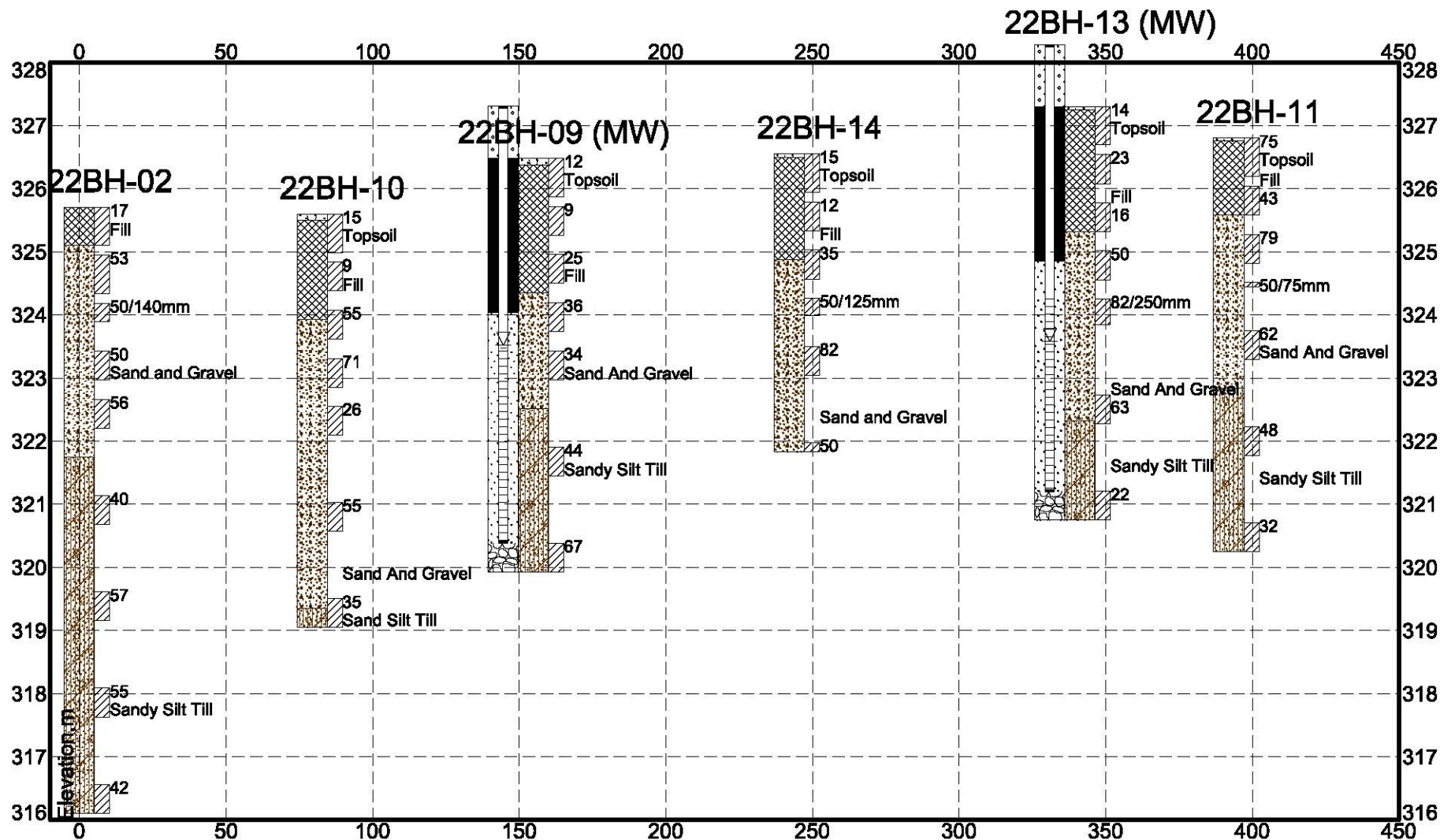
SUBSURFACE STRATIGRAPHY

Section 1

Geotechnical Investigation

115 Watson Parkway North  
(& Starwood Drive), Guelph, Ontario

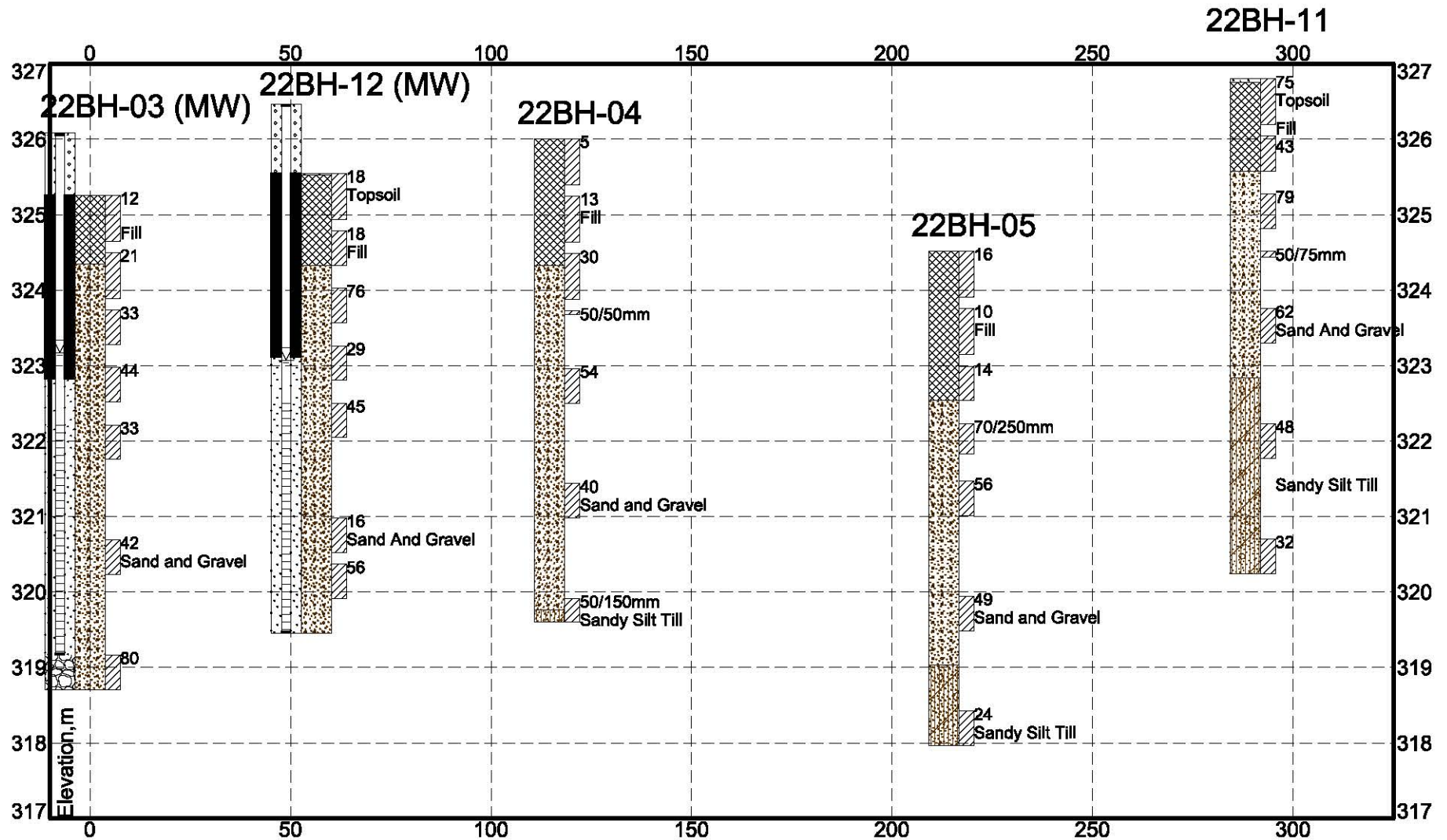
PROJECT #	DATE	DRAWING
4515-22-GC	Jun 22	18



Borehole No	Elev.	Depth
22BH-02	325.7	9.6
22BH-09 (MW)	326.5	6.6
22BH-10	325.6	6.6
22BH-11	326.8	6.6
22BH-13 (MW)	327.3	6.6
22BH-14	326.6	4.7

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SUBSURFACE STRATIGRAPHY		
Section 2		
Geotechnical Investigation		
115 Watson Parkway North ( & Starwood Drive), Guelph, Ontario		
PROJECT #	DATE	DRAWING
4515-22-GC	Jun 22	19



Borehole No	Elev.	Depth
22BH-03 (MW)	325.3	6.6
22BH-04	326.0	6.4
22BH-05	324.5	6.6
22BH-11	326.8	6.6
22BH-12 (MW)	325.6	6.1

Toronto Inspection Ltd.

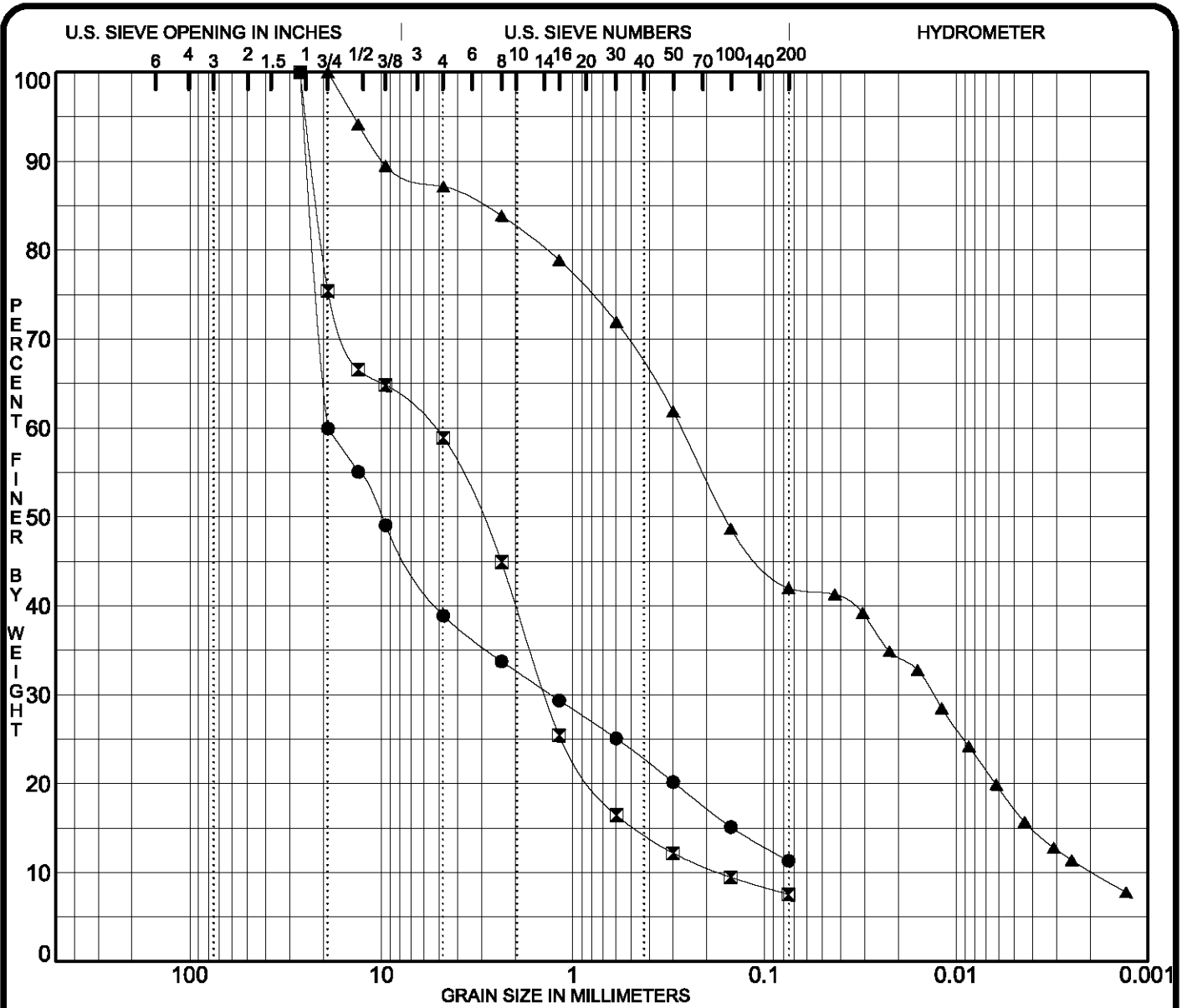
**SUBSURFACE STRATIGRAPHY**

**Section 3**

**Geotechnical Investigation**

115 Watson Parkway North  
(& Starwood Drive), Guelph, Ontario

PROJECT #	DATE	DRAWING
4515-22-GC	Jun 22	20



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

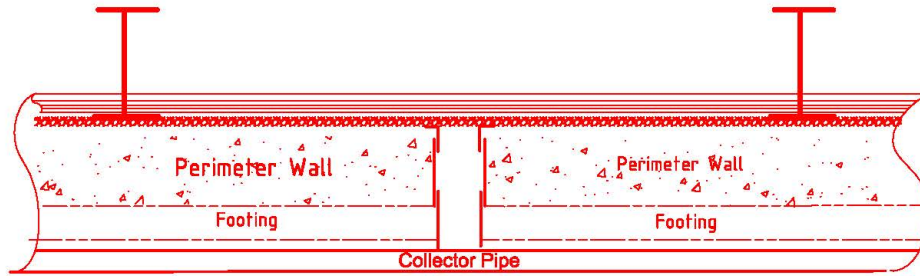
Specimen Identification	Classification					MC%	LL	PL	PI	Cc	Cu
● 22BH-12 (MW)3.0										1.52	323.6
⊠ 22BH-4 4.6										2.08	31.4
▲ 22BH-8 (MW)12.2										0.33	140.2
Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay			
● 22BH-12 (MW)3.0	26.50	19.01	1.303		61.1	27.6	11.3				
⊠ 22BH-4 4.6	26.50	5.39	1.389	0.1716	41.1	51.4	7.5				
▲ 22BH-8 (MW)12.2	19.00	0.27	0.013	0.0019	12.8	45.2	24.7	17.3			

PROJECT **Geotechnical Investigation - Starwood Drive & Watson Parkway North, Guelph, Ontario**

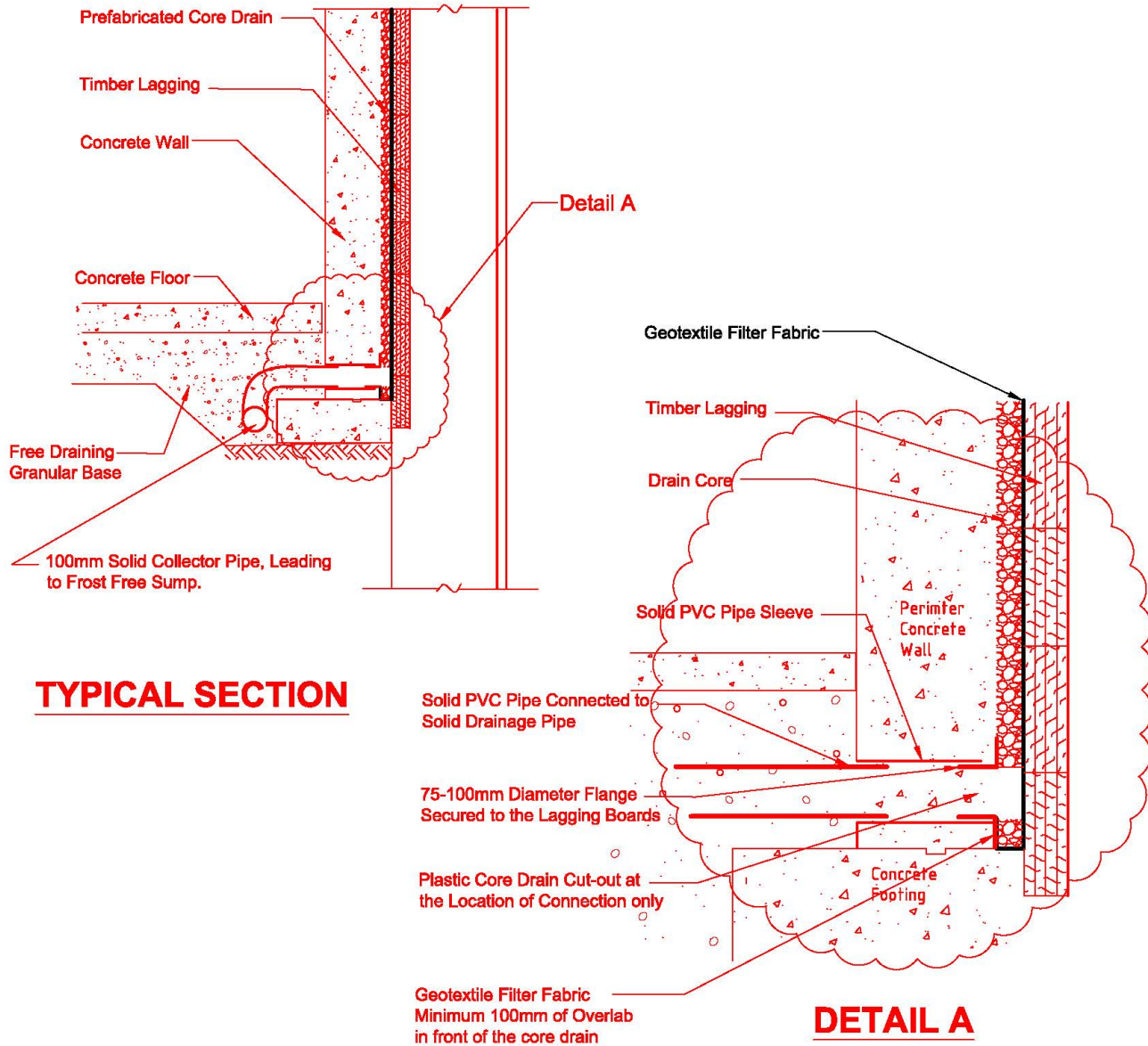
JOB NO. **4515-22-GC**  
DATE **6/6/22**

**GRADATION CURVES**  
Toronto Inspection Ltd.

**FIGURE NO. 1**



## PLAN



## TYPICAL SECTION

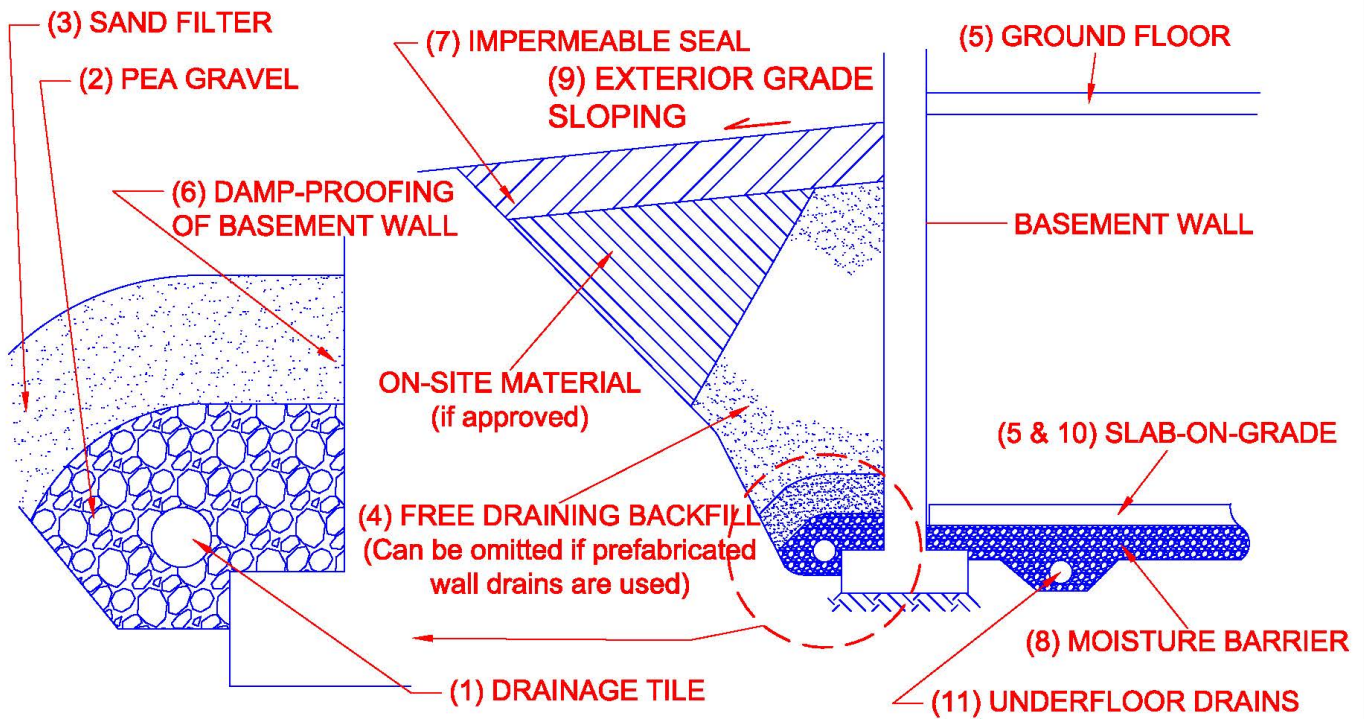
## DETAIL A

### Note:

1. A continuous blanket of prefabricated drainage system, Miradrain 6000 or equivalent, should extend continuously from the top of footings to approximately 1.2m below the ground surface.
2. All joints of the Miradrain should be taped. All openings, including the exposed end above the footing, must be covered with filter fabric to prevent intrusion of concrete into the core of the drain.
3. The backfill behind the lagging must be free draining. Filter fabric or straw should be used to prevent loss of fines behind the lagging.
4. The perimeter drainage and subfloor drainage systems must be kept separate.

NOT TO SCALE





### Notes:

1. **Drainage tile:** consist of 100mm (4") diameter weeping tile or equivalent perforated pipe leading to a positive sump or outlet. invert to be at minimum of 150mm (6") below underside of basement floor level.
2. **Pea gravel:** at 150mm (6") on the top and sides of drain. If drain is not placed on footing, provide 100mm (4") of pea gravel below drain. The pea gravel may be replaced by 20mm clear stone provided that the drain is covered by a porous geotextile membrane of Terrafix 270 R or equivalent.
3. **Filter material:** consists of C.S.A. fine concrete aggregate. A minimum of 300mm (12") on the top and sides of gravel. This may be replaced by an approved porous geotextile membrane of Terrafix 270R or equivalent.
4. **Free-draining backfill:** OPSS Granular B or equivalent, compacted to 93 to 95% (maximum) Standard Proctor Density. Do not compact closer than 1.8m (6ft.) from wall with heavy equipment. This may be replaced by on site material if prefabricated wall drains (Miradrain) extending from the finished grade to the bottom of the basement wall are used.
5. **Do not backfill** until the wall is supported by the basement floor slab and ground floor framing, or adequate bracing.
6. **Damp-proofing** of the basement wall is required before backfilling.
7. **Impermeable backfill seal** of compacted clay, clayey silt or equivalent. If the original soil in the vicinity is a free draining sand, the seal may be omitted.
8. **Moisture barrier:** consists of 20mm clear stone or compacted OPSS Granular A, or equivalent. The thickness of this layer to be 150mm (6") minimum.
9. **Exterior Grade:** slope away from basement wall on all the sides of the building.
10. **Slab-on-grade** should not be structurally connected to walls or foundations.
11. **Underfloor drains** \* should be placed in parallel rows at 6-8m (20-25 ft.) centre, on 100mm (4") of pea gravel with 150mm (6") of pea gravel on top and sides. The invert should be at least 300mm (12") below the underside of the floor slab. The drains should be connected to positive sumps or outlets. Do not connect the underfloor drains to the perimeter drains.

\* Underfloor drains can be deleted where not required.

NOT TO SCALE



**Toronto Inspection Ltd.**

**APPENDIX A**  
***Shoring Design***

## APPENDIX A SHORING DESIGN

All specifications for the design of the shoring system are in accordance with Chapter 26 of the 4th Edition of the Canadian Foundation Engineering Manual (Manual).

The construction of the shoring system should be carried out by a contractor experienced in this type of construction.

### 1. Earth pressure

For a single and multiple level support systems, the recommended earth pressure distributions are shown on Drawing A1.

The lateral earth pressure expressions, recommended in the drawings, assume that there will be no build up of hydrostatic pressure behind the shoring.

### 2. Pile Penetration

The soldier piles should be installed in pre-augured holes which should be filled to excavation level with 20 MPa (3000 psi) concrete and above that with 1-1/2 bag mix.

The depth of pile penetration in the non-cohesive sand and gravel to sandy silt till deposits should be calculated from the following expressions:

$$R \text{ (sand and gravel to sandy silt till) } = 1.5 D K_p L^2 \gamma$$

where	R = Ultimate Load to be restrained	kN
	D = Diameter of concrete filled hole	m
	K <sub>p</sub> = Passive resistance in the sandy silty till deposit	5.0
	L = Embedment Depth of the pile	m
	γ = Unit weight of the soil - use 21 kN/m <sup>3</sup> for unsaturated soils	

The shoring system should be designed for a factor of safety of F = 2. The overall factor of safety of the anchored block of soil must be considered.

### 3. Lagging Boards

The following thicknesses of lagging boards have been recommended in the Manual:

<u>Thickness of lagging</u>	<u>Maximum Spacing of Soldier Piles</u>
50 mm ( 2 in )	2.0 m (6.5 ft )
75 mm ( 3 in )	2.5 m ( 8.0 ft )
100 mm ( 4 in )	3.0 m ( 10 ft )

Local experience has indicated that the lagging thickness of 75 mm has been adequate for soldier pile spacing of 3 m for soil conditions similar to those encountered at the subject site. However, it is important to consider all local conditions, such as the duration of excavation, the weather likely to be encountered, seasonal variations in the ground water and ice lensing causing frost heave in determining the lagging thickness.

**All spaces behind the lagging must be filled with free draining granular fill.** If wet conditions are encountered the space between boards should be packed with geotextile filter fabric or straw to prevent loss of ground.

#### **4. Tie Backs**

The minimum spacing and the depths of the soil anchors should be as recommended in the Manual.

The tie back anchor lengths, in the non-cohesive sand and gravel to sandy silt till deposits, can be estimated using an adhesion values of 50 kPa (1000 psf). At least two full scale load tests should be carried out on the tieback anchors in each of the above subsoils. These tests should be taken to 200% of the design load or until there is a significant increase in the pullout rate. In the latter case, the design load must be limited to 50% of the load at which the pullout increases. Based on the results of the pullout test, it may be necessary to modify the anchor design and place limits on the capacity.

In addition, each anchor must be proof loaded. This is done by loading the anchor to 133% of the design load, and the anchor must be capable of sustaining this load for a minimum of 10 minutes without creep. The load may then be relaxed to 100% of design and locked in. The higher the lock in loads, the less will be the outward movement after excavation.

The proposed design of the tie-back system and method of installation must be discussed with this office prior to the finalization. Systems involving high grout pressures should be avoided if working near other basements or buried services.

#### **5. Rakers**

An alternative to tie backs is to use rakers. Rakers founded in sand and gravel to sandy silt till deposits should be designed for allowable bearing pressures of 150 kPa (3.0 k.s.f.), for rakers inclined at an angle of 45 degrees.

The raker footings should be located outside the zone of influence of the buried portion of the soldier piles and at a distance of not less than 1.5 L from the piles, where L = the embedment of the pile. No excavation should be made within two footing width of the raker footings on the side opposite the rakers.

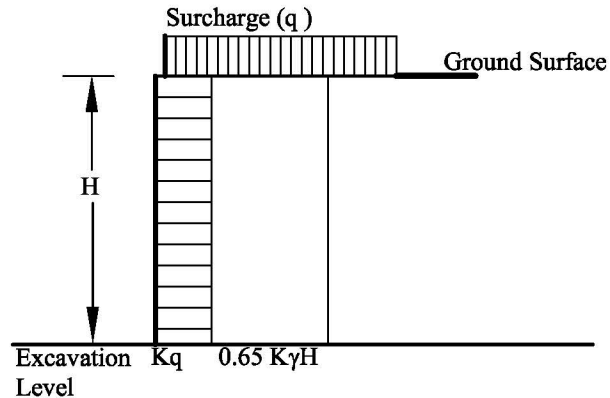
#### **6. General Shoring Notes**

It is recommended that close monitoring of vertical and lateral movement of the shoring system should be carried out at the site. If movements at the top of the piles are more than 12 mm (0.5 in), extra bracing may be required. In this regard, monitoring by inclinometers and by survey on targets should be instituted to ensure that the contractor maintains movements within design limit.

# TEMPORARY SHORING

## Lateral Pressure

### Multiple or Single Level Support



$$\text{Lateral Pressure } P = 0.65K(\gamma H) + Kq$$

where  $H$  = Height of Shoring  $\text{m}$   
 $\gamma$  = Unit Weight of Retained Soil  $21.0 \text{ kN/m}^3$   
 $q$  = Surcharge  $\text{kPa}$   
 $K$  = Earth Pressure Coefficient

If moderate ground and shoring movements are permissible then:

$$K = K_a = \text{Active Earth Pressure Coefficient} = 0.25$$

If there are building foundations or underground services within a distance of  $0.5 H$  behind the shoring then:

$$K = K_o = \text{Earth Pressure at rest} = 0.4$$

If there are building foundations or underground services within a distance of between  $0.5 H$  and  $H$  behind the shoring then:

$$K = 0.5 (K_a + K_o) = 0.33$$

Note:

The lateral pressure equation assumes effective drainage from behind the temporary shoring

NOT TO SCALE