REPORT ON
GEOTECHNICAL INVESTIGATION
48 ISABELLA STREET
TORONTO, ONTARIO

REPORT NO.: 6793-24-GA
REPORT DATE: JANUARY 24, 2025

PREPARED FOR
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1.0 INTRODUCTION

Toronto Inspection Ltd. (TIL), was authorized by Land's Edge Properties Ltd. to conduct a geotechnical Investigation for the proposed redevelopment at the property, located at 48 Isabella Street, in Toronto, Ontario (hereinafter referred to as “the Site”). The field work for the geotechnical investigation was carried out in conjunction with a Hydrogeological study. The reports of findings, relating to the Hydrogeological study, will be issued under a separate cover.

A set of Architectural drawings, prepared by Kirkor Architects & Planners, dated September 24, 2024, received from the client, indicated that the proposed redevelopment of the Site will consist of a 68 storey residential building, with four levels of underground parking (at a depth of 12.0m below ground floor level).

The purpose of the geotechnical investigation was to delineate the subsoil and groundwater conditions, encountered at the borehole locations, and provide our assessment for the design and construction of the redevelopment. In particular, geotechnical data was to be provided for:

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

This geotechnical investigation 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 0.17 ha in area and rectangle in shape, is located on the north side of Isabella Street, on the south side of Macy Dubois Lane, approximately 180m east of Yonge Street, in Toronto, Ontario.

At the time of the investigation, the Site was occupied by a 10 storey brick apartment building with one level of underground parking. The Site gradient was relatively flat and slightly higher than Isabella street, but slightly lower than Macy Dubois Lane.

3.0 INVESTIGATION PROCEDURE

The field work for the investigation was carried out during the period of November 22 to December 6, 2024, and consisted of drilling five sampled boreholes (24BH-1 to 24BH-5). Three of boreholes, 24BH-1, 24BH-2 and 24BH-5, drilled in the one level of underground parking, extended to depths of 12.6m, 21.5m and 21.6m from grade, respectively; the remaining boreholes, 24BH-3 and 24BH-4, extended to depths of 15.7m and 24.8m from grade, respectively; at the locations shown on the appended Borehole Location Plan (Drawing No. 1). Due to limited access to the drill rig, no boreholes could be drilled at the southwest portion of the Site.

The borehole was advanced using a Hilti and track mounted drill rig, equipped with continuous flight hollow stem augers and sampling rods, with mud rotary, supplied by a specialist drilling contractor. Soil samples were retrieved from the boreholes at 0.76m intervals to depths of 3m below the existing ground level. Below these depths, the sampling frequency was increased to 1.5m. The samples were obtained using a split spoon sampler in conjunction with Standard Penetration Tests (SPT) using a driving energy of 475 joules (350 ft-lbs). The samples were identified and logged in the field and were carefully bagged and delivered to our laboratory for moisture content determination and visual identification by a geotechnical engineer.

Groundwater observations were made in the open borehole during and upon the completion of drilling, when applicable. All boreholes, 24BH-1 to 24BH-5, were completed as monitoring wells for the determination of the current groundwater conditions. The symbol (MW), beside the borehole identification, indicates a monitoring well. The groundwater records are presented in the borehole logs.

The borehole locations, established in the field by our site personnel, are shown on the appended Borehole Location Plan (Drawing No. 1). The ground elevations at the borehole locations were obtained by interpolation of the spot elevations, shown on a Plan of Survey, Showing Topography of Part of Park Lots 7 and 8, Concession 1, From the Bay, City of Toronto, prepared by J.D. Barnes Limited, dated October 2, 2024, provided to our office by the client. However, the ground elevations at Boreholes 24BH-1, 2BH-2 and 24BH-5 were determined related to the ground elevation at the entrance of the one level of underground parking.

4.0 SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Borehole Location Plan (Drawing No. 1) and Logs of Borehole sheets (Drawing Nos. 2 to 6), and a section (Drawing No. 7), for details of field work, including soil classification, inferred stratigraphy, ground water observations carried out during and on completion of the borehole.

The subsoil, below the surface course of concrete slab or topsoil, consisted of a layer of fill overlying native deposits of clayey silt, silt to sandy silt or to silty sand, sandy silt till, and silty sand.

Brief descriptions of the subsurface materials, encountered at the borehole location, are as follows:

4.1 Surface Course

Concrete slab, approximately 160mm to 170mm in thickness, over a granular base course, extending to depths of 0.25m from top of the concrete, was contacted at Boreholes 24BH-1, 24BH-2 and 24BH-5 locations.

Topsoil, approximately 150mm in thickness, was contacted at Boreholes 24BH-3 and 24BH-4 locations.

4.2 Fill

Below the surface course of the concrete slab and topsoil, a layer of fill was contacted at all borehole locations, at depths of 0.15m to 0.25m from grade. The fill consisted of a mixture of sand, silty sand, sandy silt, clayey silt and trace gravel. The fill extended to depths of 0.6m to 2.9m from grade.

4.3 Clayey Silt

A clayey silt deposit was contacted below the fill at all borehole locations, at depths of 0.6m to 2.9m from grade. The deposit contained thin layers of silt or silt till, trace to some gravel, trace to some sandy silt and seams of fine sand. The clayey silt deposit extended to depths of 2.9m to 8.8m from grade.

Based on the Standard Penetration N-values, in the range of 8 to 29 blows per 0.3m penetration, the consistency of the deposit was firm to very stiff.

The in-situ moisture contents of the soil samples from the deposit varied from 11% to 28%, indicating moist to very moist conditions.

A lower clayey silt deposit was contacted below a sandy silt/till deposit and a silty sand deposit at Boreholes 24BH-1 and 24BH-2 locations, at depths of 8.8m and 21.0m from grade, respectively. The deposit contained trace gravel, trace sandy silt, and seams of fine sand or silt.

Boreholes 24BH-1 and 24BH-2 were terminated in the lower clayey silt deposit at depths of 12.6m and 21.6m from grade, respectively.

Based on the Standard Penetration N-values, in the range of 28 to more than 100 blows per 0.3m penetration, the consistency of the lower deposit was very stiff to hard.

The in-situ moisture contents of the soil samples from the deposit varied from 12% to 16%, indicating moist to very moist conditions.

A combination of grain size analysis, determined using both mechanical sieves and hydrometer method, and Atterberg limit tests were conducted on one selected soil sample, obtained from 24BH-1 (SS2 – at a depth of 0.8m). The results of the grain size distribution and Atterberg Limits tests are shown on the appended Figures No. 1 and No. 2.

4.4 Silt / Sandy Silty / Sandy Silt Till

Inter-layered silt / sandy silt / sandy silt till deposits were contacted below the clayey silt deposit at all borehole locations, at depths of 2.9m to 8.8m from grade. The deposits predominately consisted of silt, with layers of sandy silt to sandy silt till, and contained trace to some gravel or silty sand, trace to some clayey silt, and seams of fine sand or clay.

The sandy silt / sandy silt till deposit at Boreholes 24BH-1 and 24BH-2 locations extended to depths of 8.8m and 18.0m from grade, respectively. The silt / sandy silt and the underlying sandy silt till deposit at Borehole 24BH-3 location extended to a depth of 14.9m from grade. The sandy silt / till deposit at Borehole 24BH-4 location extended to a depth of 19.5m from grade. The silt to sandy silt till deposit at Borehole 24BH-5 location extended to a depth of 16.5m from grade.

Based on the Standard Penetration N-values, in the range of 25 to more than 100 blows per 0.3m penetration, the relative density of the deposits was compact to very dense, generally in the dense state.

The in-situ moisture content of the soil samples from the deposits, varied from 8% to 28%, indicating moist to very moist, with wet pockets or layers.

A combination of grain size analysis, determined using both mechanical sieves and hydrometer method, and Atterberg limit tests were conducted on one selected soil sample, obtained from 24BH-5 (SS10 – at a depth of 10.7m). The results of the grain size distribution and Atterberg Limits tests are shown on the appended Figures No. 1 and No. 2.

4.5 Silty Sand

A silty sand deposit was contacted below the sandy silt / till deposits, at Boreholes 24BH-2 to 24BH-5 locations, at depths of 14.9m to 19.5m from grade. The deposit contained trace gravel, trace to some sandy silt, with occasional clayey silt.

Boreholes 24BH-3 to 24BH-5 were terminated in the silty sand deposit at depths of 15.7m to 24.8m from grade. The silty sand deposit at Borehole 24BH-2 location extended to a depth of 21.0m from grade.

Based on the Standard Penetration N-values, in the range of 21 to more than 100 blows per 0.3m penetration, the relative density of the silty sand deposit was generally very dense, with a compact layer at Borehole 24BH-4 at a depth of 23.0m from grade, which could be due to loosening of the non-cohesive subsoil by the water pressure.

The in-situ moisture contents of the soil samples, retrieved from the silty sand deposit, varied from 12% to 23%, indicating very moist to wet conditions.

4.6 Groundwater

Free water was recorded in the open borehole 24BH-3 at a depth of 13.41m from grade, upon completion of drilling. However, free water or cave-in could not be documented accurately at the remaining boreholes, due to use of mud rotary drilling method, for advancing the boreholes.

On January 15, 2025, the groundwater level, measured in the monitoring wells installed at Boreholes 24BH-1 to 24BH-5 location, are listed below:

BH/WELL ID	Ground Elevation	Groundwater Measured Depths / Elevations			
		Completion	Jan 15, 25	Elevation*	Remark
24BH-1 (MW)	109.48m	NA	7.75m	101.73m	-
24BH-2 (MW)	109.51m	NA	8.71m	100.80m	-
24BH-3 (MW)	112.57m	13.41m	11.68m	100.89m	-
24BH-4 (MW)	112.67m	NA	11.97m	100.70m	-
24BH-5 (MW)	109.56m	NA	7.22m	102.34m	-

NA: due to mud rotary *: Static water level after bailing out water

Based on the moisture content profile of the soil samples retrieved from the boreholes and our field observation at the Site and the groundwater records, it is our opinion that the water levels recorded represent water in the silty sand deposit and in the seams of fine sand within the silt till and clayey silt deposits. There is a probability that the water level in the silty sand deposit is under a sub-artesian condition.

The groundwater will be subject to seasonal fluctuation. The static groundwater table conditions should be rechecked to confirm current stabilized static elevations and confirmed by the hydrogeological study.

5.0 RECOMMENDATIONS

A set of Architectural drawings, prepared by Kirkor Architects & Planners, dated September 24, 2024, received from the client, indicated that the proposed redevelopment of the Site will consist of a 68 storey residential building, with four levels of underground parking (at a depth of 12.0m below ground floor level).

The proposed ground floor elevation and the slab-on-grade elevations of the underground parking were not known at the time of preparation of this report. However, we have assumed that the proposed ground floor elevation will be at or close to the existing ground floor level (at an elevation of 112.68m) and the slab-on-grade will be at a depth of 12.0m below the proposed ground floor level (at an elevation of 100.68m), for the four levels of underground parking; approximately at a depth of 8.81m from grade of P1 at Borehole 24BH-1 location. The founding levels of the spread footings are assumed to be 1.0m lower than the above assumed slab-on-grade depth, i.e. at or below depths of 13.0m below the existing ground level (at an elevation of 99.68m). However, the elevator and the surrounding foundations are anticipated to be deeper than the above assumed founding levels, at depths of approximately 15.0m below the existing ground level (at an elevation of 97.68m).

The slab-on-grade is at or slightly lower than the documented groundwater levels. The assumed foundation depths are approximately 1.0m to 4.7m below the wet conditions / groundwater levels. Unless a permanent groundwater control system is used to maintain the water level a minimum of 0.5m below the proposed lowest slab-on-grade elevation, we recommend that the part of the underground parking, below one metre above the current documented 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 structures, to resist uplift pressure of almost 65 kPa at the around the elevator shaft, for four levels of underground parking.

The recommendations provided in this report, for the design and construction of the redevelopment, are based on the subsoil and groundwater conditions encountered at the borehole locations and the assumed slab-on-grade depth of the four level underground parking, and on the assumption that the groundwater table will be maintained below the slab-on-grade. If there is a change in the depths assumed, the report will have be revised.

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

5.1 Foundations

The subsoils at and below the assumed founding depths of 8.81m to 12.0m from grade, elevations of 97.68m to 99.68m, are anticipated to consist of compact to dense sandy silt / till / silty sand, and very stiff clayey silt deposits, at the borehole locations.

Conventional spread and strip footings, founded in the undisturbed compact to dense and very stiff deposits, for the four levels of underground parking, at or below depths of 8.81m to 12.0m from grade, elevations of 97.68m to 99.68m, at the borehole locations, can be designed using the following bearing pressures, provided that the water table is maintained a minimum of 1 m below the deepest foundation level:

- 300 kPa at Serviceability Limit State (SLS)
- 450 kPa at Factored Ultimate Limit State (ULS)

The above bearing pressures are not adequate to place the proposed building on spread and strip footings. Consideration should be given to using a combination of a raft foundation with the bearing pressure of 300 kPa at SLS and deep foundations, consisting of continuous flight auger (CFA) cast piles. For the 600mm diameter CFA piles, spaced at or more than 3D, founded at or below elevation of 90.0m, approximately 10.68m below the slab-on-grade, at Boreholes 24BH-2, 24BH-4 and 24BH-5 locations, in the dense to very dense silty sand deposit, the preliminary axial load capacity of 1000 KN should be assumed.

A full scale load test will have to be carried out on a CFA to confirm this load capacity.

Due to limited access to the drill rig, no borehole could be drilled at the southwest portion of the Site. We recommend that additional boreholes will have to be carried out at the southwest portion when the access to a drill rig is available to delineate and confirm the subsoil and groundwater conditions.

The Modulus of Subgrade Reaction for the raft foundation, founded on the very stiff clayey silt and dense sandy silt/till and silt is recommended as 30 MN/m3.

If the final slab-on-grade is below the static water level, provision will have to be made to maintain the water level below the slab and elevator shafts will have to be designed as watertight structures.

The total and differential settlement of the new foundations, under the above recommended bearing pressures at Serviceability Limit State, will not exceed 25 mm and 20 mm, respectively.

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

There is no official rule governing the footing depth for a fully enclosed unheated garage. Unmonitored experience in the past has shown that footing depths of less than the frost penetration depths 1.2 m have been adequate. For the four levels of underground parking, the interior columns / walls and the perimeter wall footings can be founded at depths of 0.8m and 0.6m 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 1.2 m should be provided. In addition, a nominal 50 mm of Styrofoam insulation should be provided under the floor slab within the close proximity to the fresh air ducts.

With the uplift pressures, there could be a major damage to the structure if the permanent drainage system fails. A hodrogeological study should be referred for the drawdown curve for long term dewatering of the silty sand layer. If the drawdown curve shows significant lowering of the water table abound the Site, there is a high probability of settlements of the structures around the Site. In this respect, ***Toronto Inspection Ltd.*** should be consulted, once the slab-on-grade elevation has been determined and the drawdown curve, based on the elevations of lowering of temporary and permanent water table.

For the construction of the raft foundation, 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. Since the founding of the raft foundation will be below the groundwater table, we recommend that the part of the structure below the highest anticipated groundwater table, as established by the hydrogeological study, should be designed as a water tight structure.

It is our opinion that the temporary dewatering, during construction period, would be to use sump pits (if the slab-on-grade is at or close to the bases of the foundations), an eductor system, deep wells, vacuum well points or a combination of these systems, after the excavation has reached approximately 1m above the current static water level. The dewatering system should be designed by the dewatering contractor to maintain the water level a minimum of 1.0m below the deepest footing level.

It should be noted that the above recommendations for the design and construction of footings have been analyzed by ***Toronto Inspection Ltd.*** from the 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 ***TIL***, to validate the information for use during construction.

5.2 Floor Slab Construction

The subsoil under the proposed slab-on-grade for four levels of underground parking is anticipated to consist of sandy silt/till, silty sand/ till deposits. Provided that the groundwater table is maintained a minimum of 0.5m below he slab-on-grade elevation, the floor slab of the proposed building can be designed and constructed as a conventional slab-on-grade method.

A granular base course, consisting of at least of 150 mm of Granular A (OPSS Form 1010) or its approved equivalent, should be provided between the subsoil and the slab-on-grade as a moisture barrier. The granular base should be compacted to at least 100% of its Standard Proctor maximum dry density. It will be necessary to install the subfloor drains. We recommend that provisions should be made in the construction budget to install the subfloor drains.

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.

5.3 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 D (stiff soil).

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

5.4 Excavation

All excavations should comply with the Ontario Occupational Health and Safety Act. Any excavation in the fill should be sloped back to a safe angle of 45 degrees or flatter.

We do not anticipated any serious groundwater problems in excavation to the depths of 8.5m from grade at Borehole 21BH-1 location. Localized seepage of water from wet sand layers or seams can be drained to sump pits and removed by pumping from sumps. Below this depth, localized de-watering system will be required in further excavation and for foundations.

In areas where adequate space will not be available for an open excavation, a temporary shoring system 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.

5.5 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
	γ = Bulk unit weight of the soil	21.5 kN/m^3
	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, 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:

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

where	P_s = Lateral earth pressure below the water table	kPa
	K = Lateral earth pressure coefficient	0.4
	γ' = Submerged unit weight of the soil	11.7 kN / m^3
	H = Depth of the wall below the water level	m
	$\gamma\omega$= Unit weight of water	9.8 kN / m3
	q = Surcharge loads adjacent to the basement wall	kPa

5.6 Permanent Perimeter Drainage

Permanent perimeter drains should be provided around the basement 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. 3. The installation of this type of vertical drainage system and its connections should be carried out as per the manufacturer's specifications.

5.7 Groundwater Control

A hodrogeological study should be referred for source of the groundwater, the groundwater table and the temporary / permanent groundwater control.

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, lost 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.

Yours very truly,
TORONTO INSPECTION LTD.

LICENSED PROFESSIONAL ENGINEER
S. WANG
100131918
Jan. 24, 2025
PROVINCE OF ONTARIO

David S. Wang, P.Eng.
Senior Engineer

REGISTERED PROFESSIONAL ENGINEER
USS
U.S. SAPPAL
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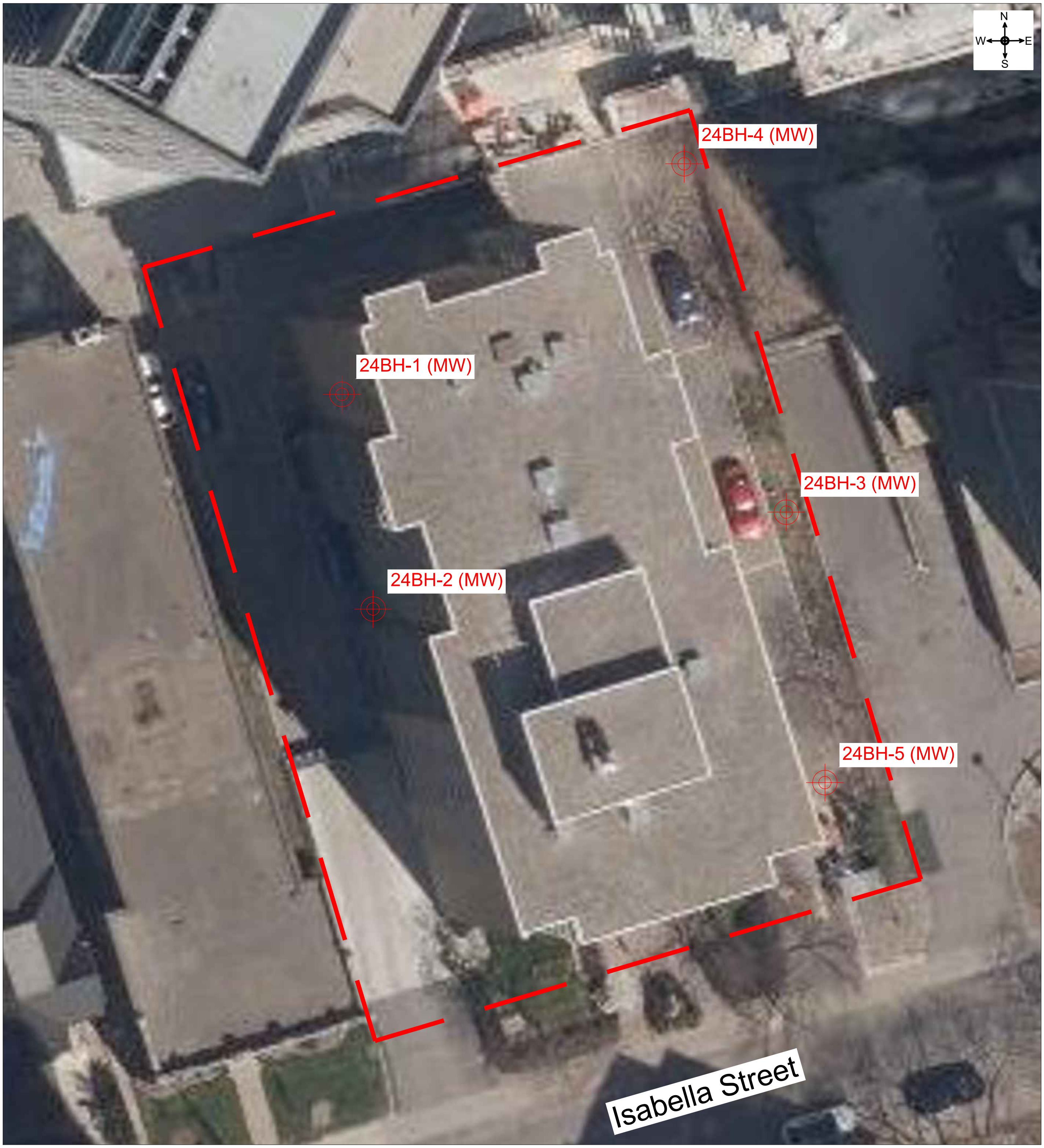
PROVINCE OF ONTARIO

Upkar S. Sappal, P.Eng.
Principal Engineer

Drawings
Borehole Location Plan
Logs of Boreholes
Section



LEGEND:

Borehole and Monitoring Well Location

Site Boundary

NOT TO SCALE

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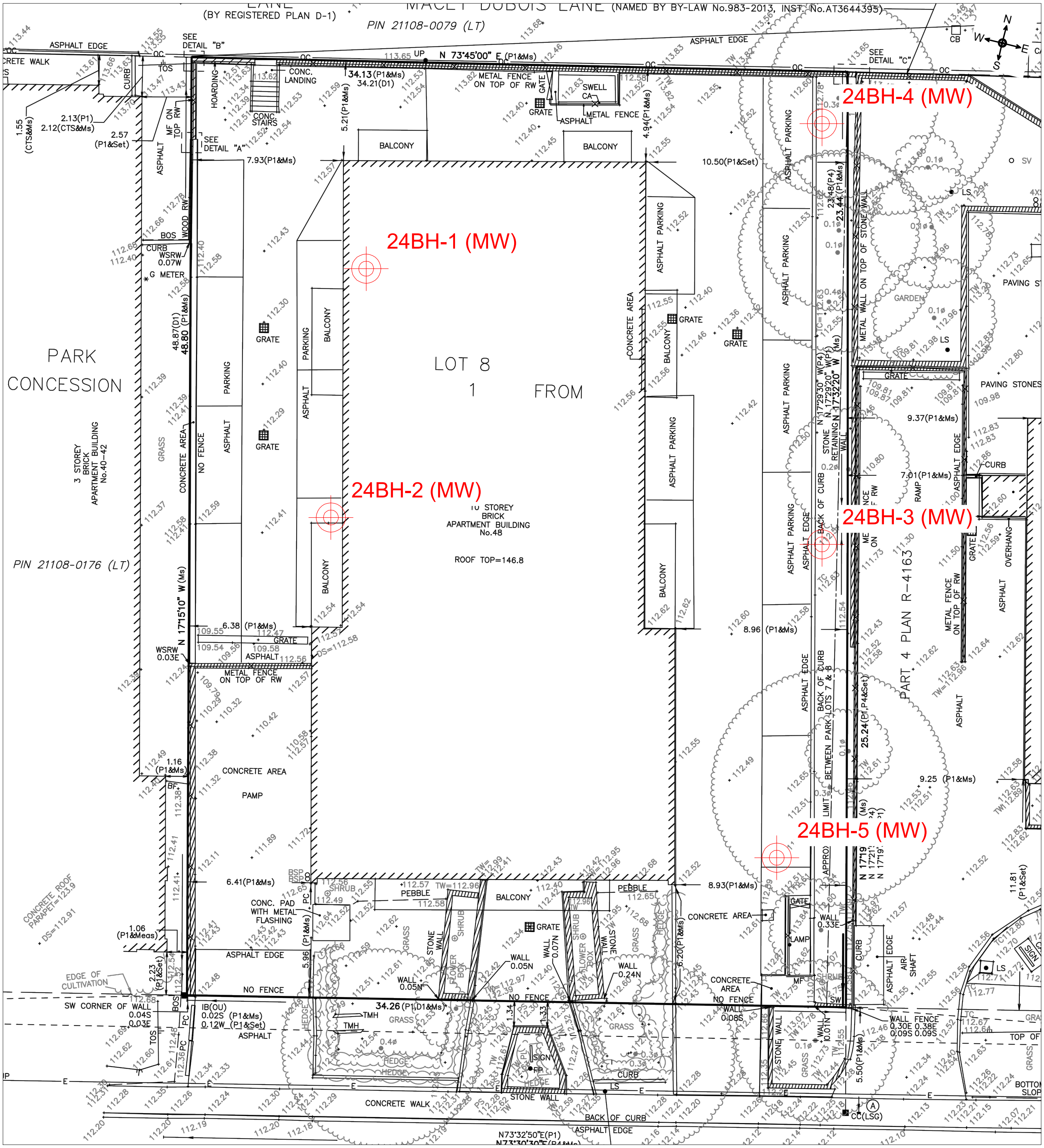
TITLE:	Borehole and Monitoring Well Location Plan
LOCATION:	48 Isabella Street, Toronto, Ontario

PROJECT NO.:	DATE :	DRAWING NO.
6793-24-GA	December 2024	1



LEGEND:

Borehole and Monitoring Well Location

NOT TO SCALE

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TITLE:	Borehole and Monitoring Well Location Plan
LOCATION:	48 Isabella Street, Toronto, Ontario
PROJECT NO.: 6793-24-GA	DATE : December 2024
DRAWING NO.	1

Project No. 6793-24-GA

Log of Borehole 24BH-1 (MW)

Dwg No. 2

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 48 Isabella Street, Toronto, Ontario

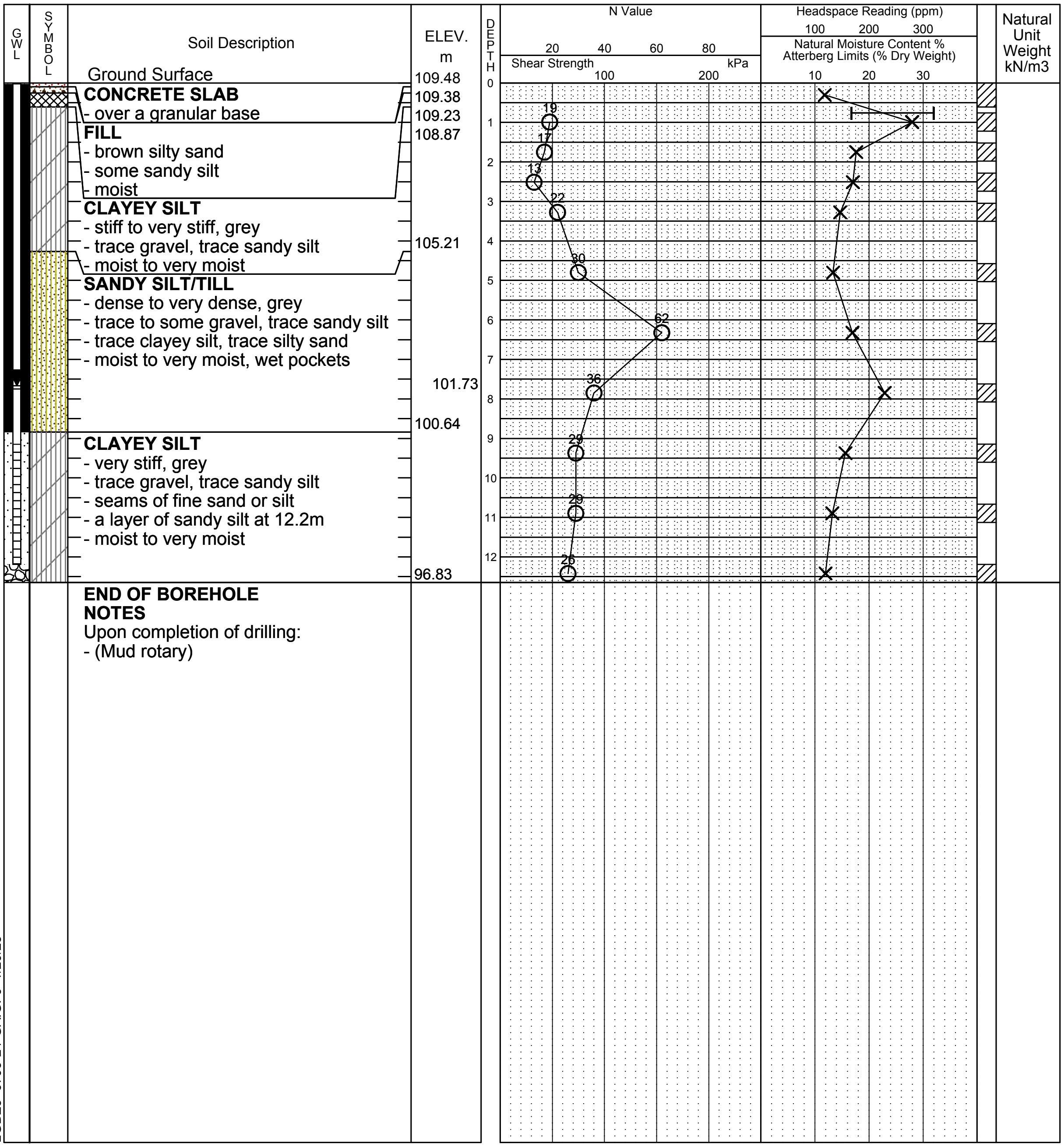
Date Drilled: 11/25/24

Drill Type: Hilti

Datum: Geodetic

Legend	
Auger Sample	⊠
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Headspace Reading (ppm)	•
Natural Moisture	✕
Plastic and Liquid Limit	├─┤
Unconfined Compression % Strain at Failure	⊗
Penetrometer	▲

Soil Description	ELEV. m
Ground Surface	109.48
CONCRETE SLAB	109.38
- over a granular base	109.23
FILL - brown silty sand - some sandy silt - moist	108.87
CLAYEY SILT - stiff to very stiff, grey - trace gravel, trace sandy silt - moist to very moist	105.21
SANDY SILT/TILL - dense to very dense, grey - trace to some gravel, trace sandy silt - trace clayey silt, trace silty sand - moist to very moist, wet pockets	101.73 100.64
CLAYEY SILT - very stiff, grey - trace gravel, trace sandy silt - seams of fine sand or silt - a layer of sandy silt at 12.2m - moist to very moist	96.83
END OF BOREHOLE **NOTES** Upon completion of drilling: - (Mud rotary)	



LGBE3 6793-24-GA.GPJ 1/20/25

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)
January 15, 2025	7.75m	

Project No. 6793-24-GA

Log of Borehole 24BH-2 (MW)

Dwg No. 3

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 48 Isabella Street, Toronto, Ontario

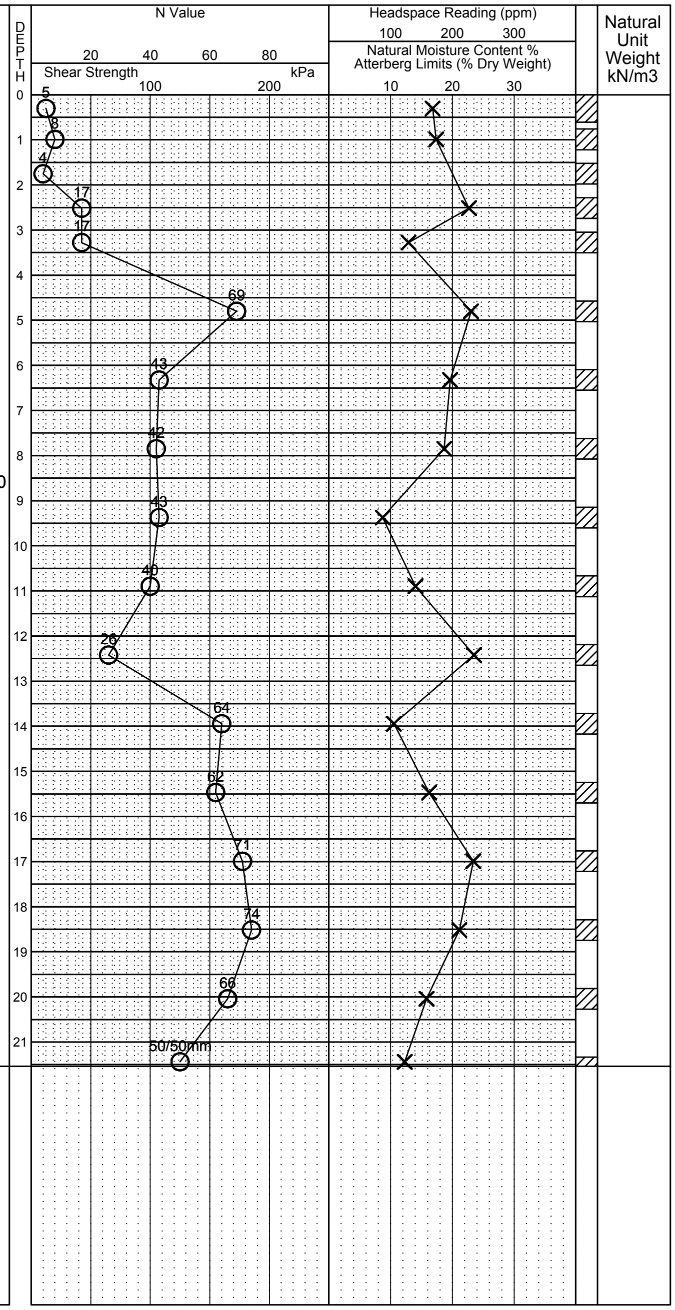
Date Drilled: 11/28/24

Drill Type: Hilti

Datum: Geodetic

Auger Sample	☒	Headspace Reading (ppm)	●
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	──	Plastic and Liquid Limit	├─┤
Shelby Tube	■	Unconfined Compression % Strain at Failure	⊗
Field Vane Test	✚ S	Penetrometer	▲

Soil Description	ELEV. m	DEPTH (m)
Ground Surface	109.51	0
CONCRETE SLAB - over a granular base	109.41 109.26	
FILL - brown silty sand to sandy silt - trace gravel, some clayey silt - moist to very moist, wet pockets	106.61	
CLAYEY SILT - very stiff, grey - trace gravel, trace sandy silt - moist	105.24	
SANDY SILTTILL - compact to very dense, grey - some gravel, trace to some clayey silt - seams of fine sand or clay - layers of sandy silt or silt - moist to very moist, wet pockets	100.80 91.53	
SILTY SAND - very dense, grey - trace gravel, some sandy silt - wet	88.48	
CLAYEY SILT - hard, grey - seams of fine sand or silt - moist	87.97	
END OF BOREHOLE **NOTES** Upon completion of drilling: - (Mud rotary)		



LGBE3 6793-24-GA.GPJ 1/20/25

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)
January 15, 2025	8.71m	

Project No. 6793-24-GA

Log of Borehole 24BH-3 (MW)

Dwg No. 4

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 48 Isabella Street, Toronto, Ontario

Date Drilled: 11/29/24

Drill Type: Track Mounted Drill Rig

Datum: Geodetic

Legend		Legend	
Auger Sample	⊠	Headspace Reading (ppm)	●
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	——	Plastic and Liquid Limit	├──┤
Shelby Tube	■	Unconfined Compression % Strain at Failure	⊗
Field Vane Test	✚ S	Penetrometer	▲

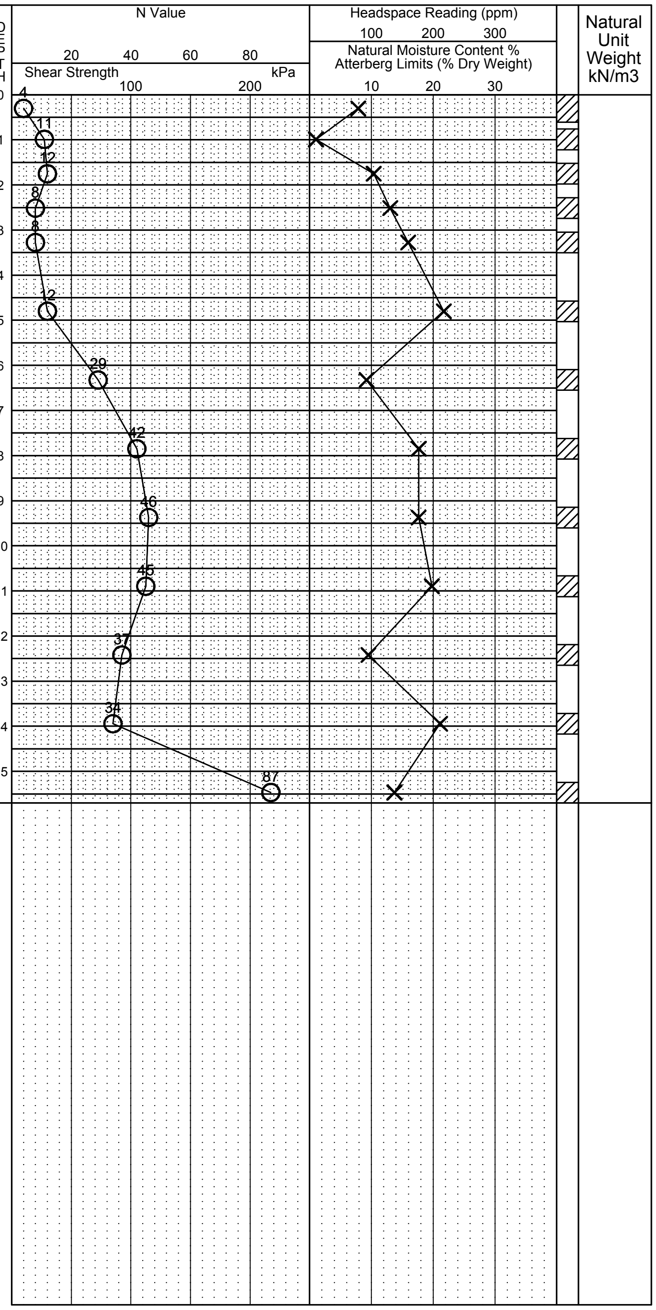
GWL	SYMBOL	Soil Description	ELEV. m	DEPTH
		Ground Surface	112.57	0
		TOPSOIL	112.42	
		FILL - brown sand - trace gravel, pockets of sandy silt - minor topsoil - moist	110.74	1–2
		CLAYEY SILT - firm, grey - trace gravel, some sandy silt - trace silty sand - moist to very moist	106.78	2–6
		SILT/SANDY SILT - compact to dense, grey - a thin layer of sandy silt till at 6.0m - trace gravel, some silty sand - very moist to wet	100.68 / 100.89	6–12
		SANDY SILT TILL - dense, grey - some gravel, some clayey silt - trace silty sand - moist to very moist	97.63	12–15
		SILTY SAND - very dense, grey - fine grained - trace sandy silt - very moist to wet	96.87	15

END OF BOREHOLE

NOTES

Upon completion of drilling:

- (Mud rotary)



LGBE3 6793-24-GA.GPJ 1/20/25

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)
January 15, 2025	11.68m	

Project No. 6793-24-GA

Log of Borehole 24BH-4 (MW)

Dwg No. 5

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 48 Isabella Street, Toronto, Ontario

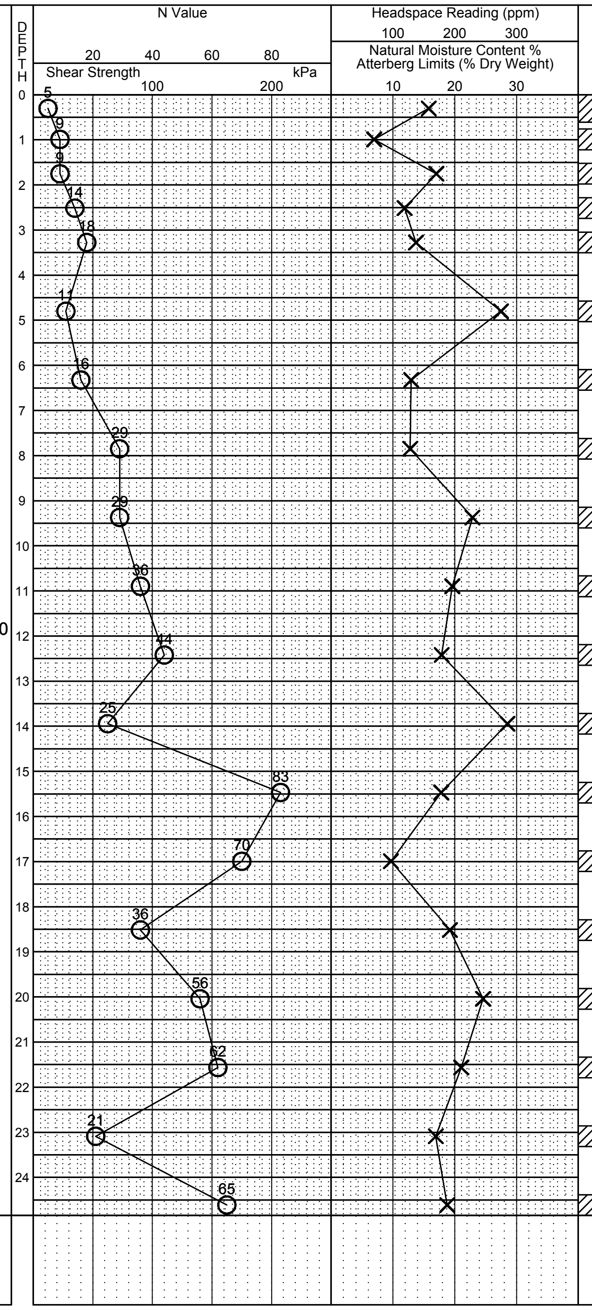
Date Drilled: 12/3/24

Drill Type: Track Mounted Drill Rig

Datum: Geodetic

Legend	
Auger Sample	⊠
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Headspace Reading (ppm)	•
Natural Moisture	×
Plastic and Liquid Limit	├─┤
Unconfined Compression % Strain at Failure	⊗
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
		Ground Surface	112.67
		TOPSOIL	112.52
		FILL - brown sand - trace gravel, pockets of sandy silt - minor topsoil or rootlets - moist	110.54
		CLAYEY SILT - stiff to very stiff, grey - trace to some gravel, some sandy silt - seams of fine sand - occasional thin alayers of clayey silt till - moist to very moist	103.83
		SANDY SILT/TILL - compact to very dense, grey - some gravel, some clayey silt - a layer of silty sand at 9.0m & 10.5m - a layer of silt at 15.0m - a layer of clayey silt at 18.0m - moist to very moist, wet layers	100.70 93.16
		SILTY SAND - very dense, compact at 22.9m - grey - trace gravel - sandy silt, some clayey silt at 24.4m - wet	87.83
		END OF BOREHOLE **NOTES** Upon completion of drilling: - (Mud rotary)	



Natural Unit Weight kN/m3

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

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Time	Water Level (m)	Depth to Cave (m)
January 15, 2025	11.97m	

LGBE3 6793-24-GA.GPJ 1/20/25

Project No. 6793-24-GA

Log of Borehole 24BH-5 (MW)

Dwg No. 6

Project: Geotechnical Investigation

Sheet No. 1 of 1

Location: 48 Isabella Street, Toronto, Ontario

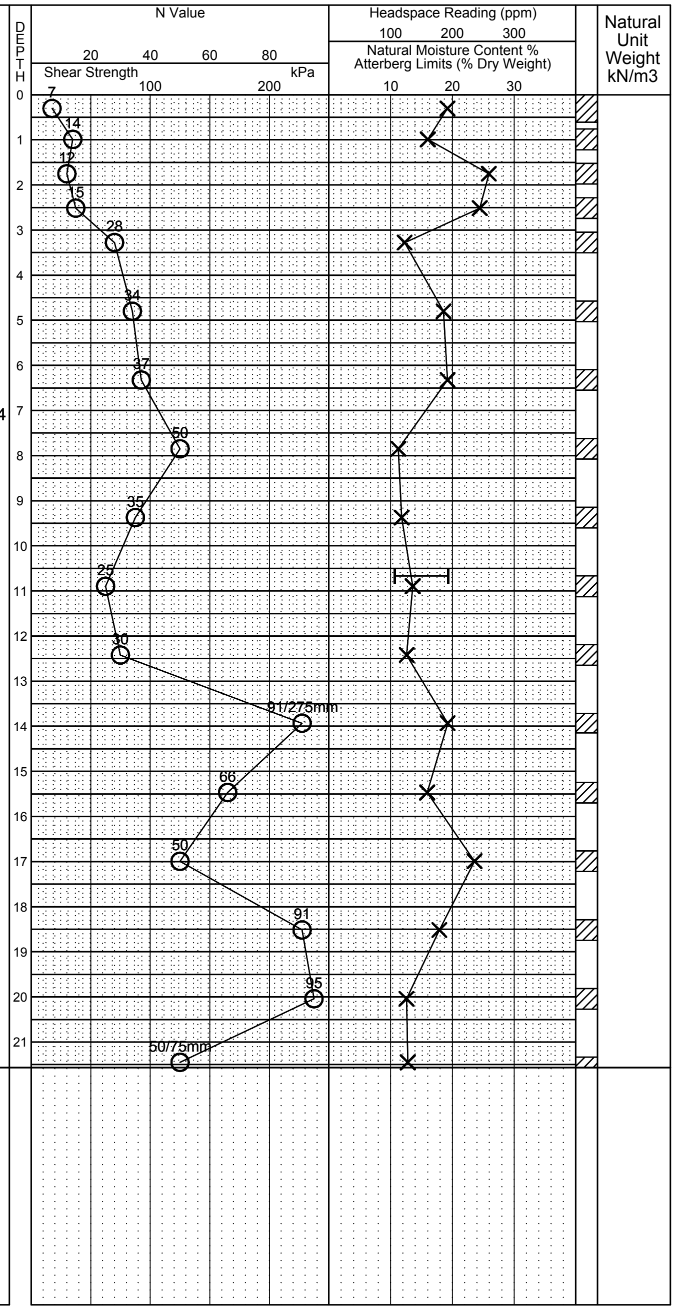
Date Drilled: 12/6/24

Drill Type: Hilti

Datum: Geodetic

Legend		Legend	
Auger Sample	⊠	Headspace Reading (ppm)	•
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	⊢—⊣
Shelby Tube	■	Unconfined Compression % Strain at Failure	⊗
Field Vane Test	✚ S	Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
		Ground Surface	109.56
		CONCRETE SLAB	109.46
		- over a granular base	109.31
		FILL - dark brown to brown clayey silt - trace gravel, pockets of sandy silt - trace gravel, some silty sand - moist	108.95 106.66
		CLAYEY SILT - stiff, grey - trace gravel, some sandy silt - trace silty sand - moist	103.77
		SANDY SILT TILL/SILT - compact to dense, grey - some gravel, some clayey silt - a layer of silt at 4.5m - moist to very moist, wet pockets	102.34
		SANDY SILT/SILT - dense to very dense, compact at 10.7m - grey - trace gravel, some silty sand - a layer of clayey silt at 10.7m - moist to very moist, wet layers	94.63
		SANDY SILT TILL - very dense, grey - trace gravel, trace clayey silt - moist	93.10
		SILTY SAND - very dense, grey - trace gravel, some sandy silt - a layer of silt at 16.8m - seams of clayey silt at 16.8m - very moist to wet	88.00
		END OF BOREHOLE **NOTES** Upon completion of drilling: - (Mud rotary)	



LGBE3 6793-24-GA.GPJ 1/20/25

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

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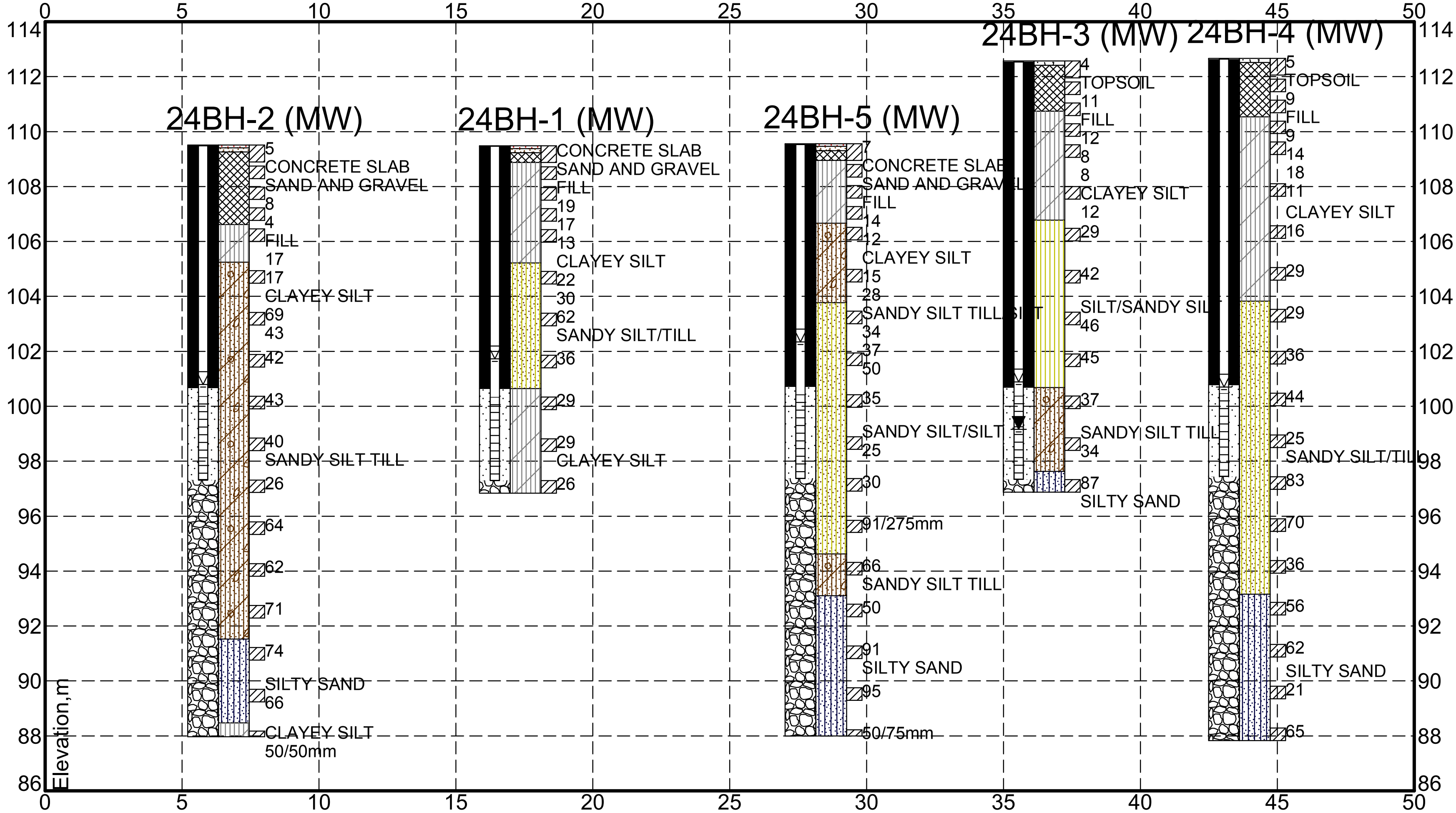
Time	Water Level (m)	Depth to Cave (m)
January 15, 2025	7.22m	



Borehole No	Elev.	Depth
24BH-1 (MW)	109.5	12.6
24BH-2 (MW)	109.5	21.5
24BH-3 (MW)	112.6	15.7
24BH-4 (MW)	112.7	24.8
24BH-5 (MW)	109.6	21.6

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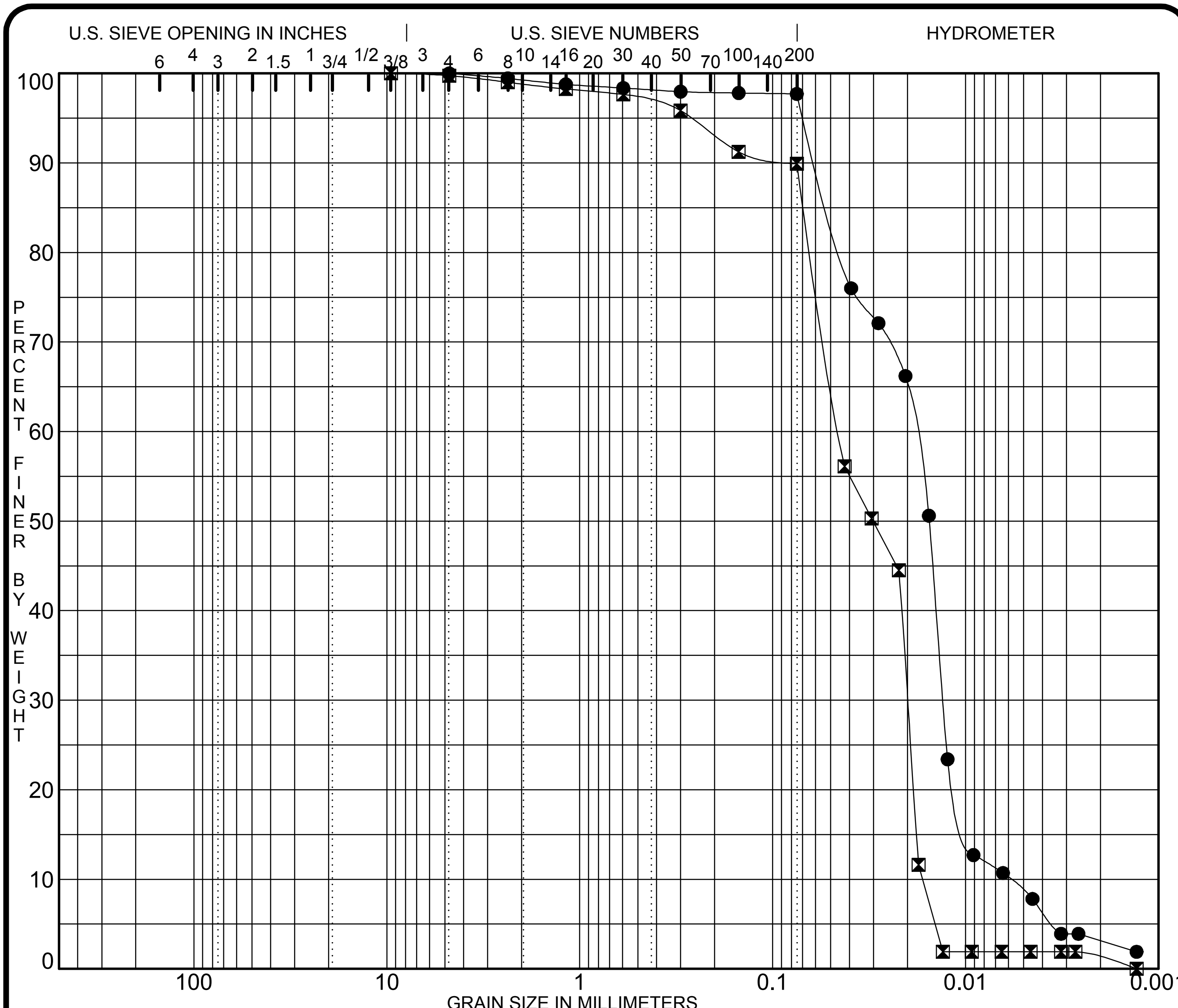
SUBSURFACE STRATIGRAPHY SECTION

Geotechnical Investigation

48 Isabella Street, Toronto, Ontario

PROJECT #	DATE	DRAWING
6793-24-GA	Jan 25	7

Figures
Gradation Curves
Atterberg Limits
Permanent Perimeter Drainage System



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**24BH-1 (MW) 0.8**	**LEAN CLAY CL**		**32**	**17**	**15**	**1.59**	**3.1**
⊠	**24BH-5 (MW)10.7**	**LEAN CLAY CL**		**19**	**11**	**9**	**0.53**	**2.7**

	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**24BH-1 (MW) 0.8**	**4.75**	**0.02**	**0.013**	**0.0059**	**0.0**	**2.3**	**89.0**	**8.7**
⊠	**24BH-5 (MW)10.7**	**9.50**	**0.05**	**0.020**	**0.0167**	**0.2**	**9.8**	**88.0**	**1.9**

PROJECT **Geotechnical Investigation - 48 Isabella Street, Toronto, Ontario**

JOB NO. **6793-24-GA**
DATE **1/8/25**

GRADATION CURVES
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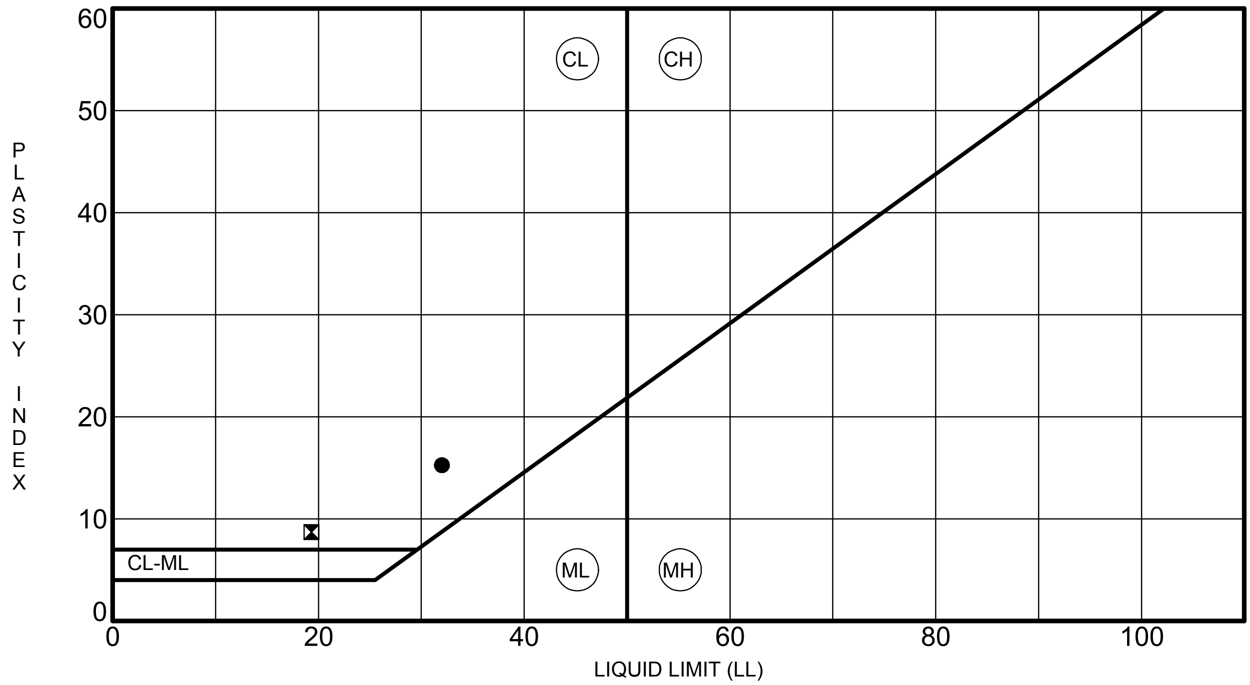
FIGURE NO. 1



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**24BH-1 (MW) 0.8**	**32**	**17**	**15**	**97.7**	**LEAN CLAY CL**
⧗	**24BH-5 (MW) 10.7**	**19**	**11**	**8**	**89.9**	**LEAN CLAY CL**

PROJECT **Geotechnical Investigation - 48 Isabella Street, Toronto, Ontario**

JOB NO. **6793-24-GA**
DATE **1/8/25**

ATTERBERG LIMITS' RESULTS **FIGURE NO. 2**

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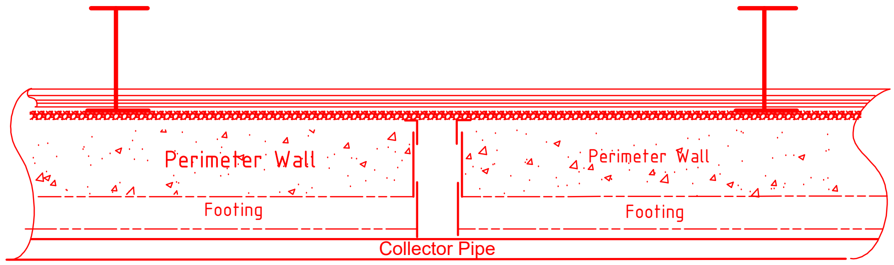
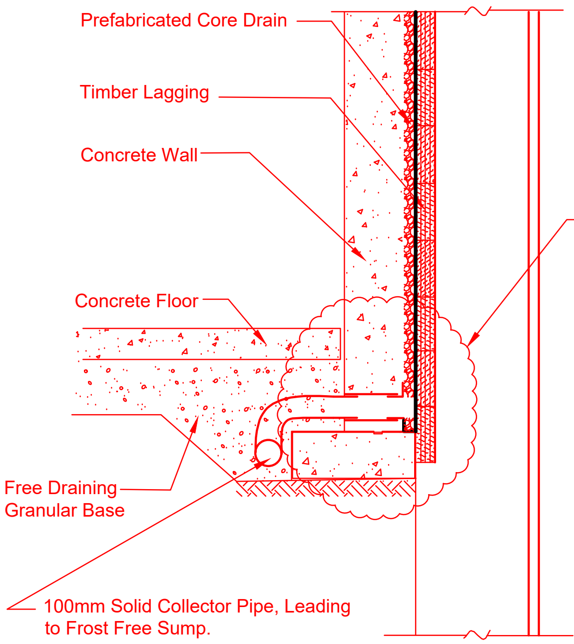
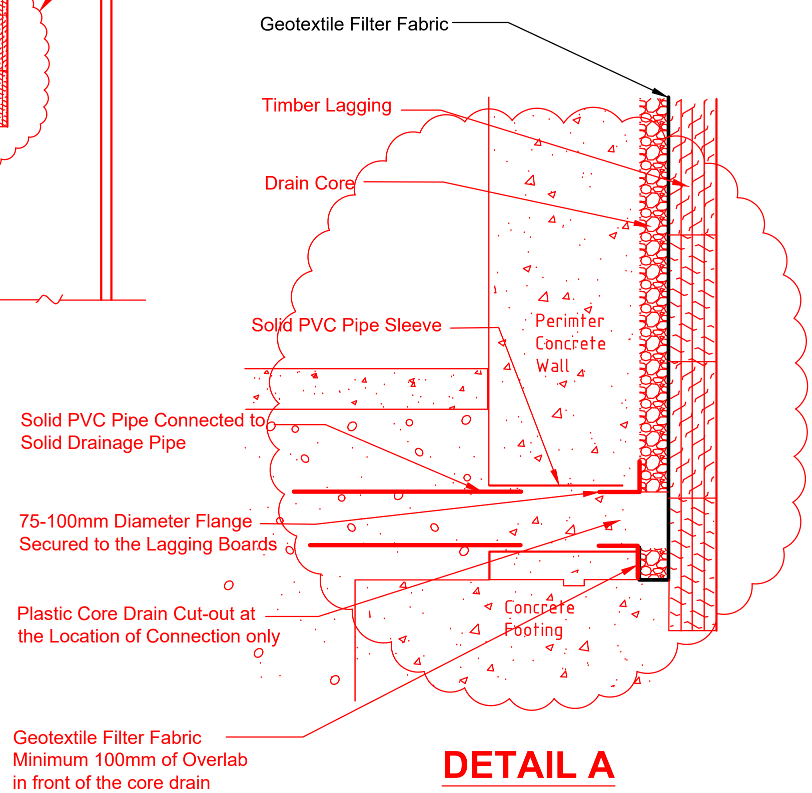


PLAN



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 fabic 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.

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TITLE:

Permanent Perimeter Drainage System

FIGURE NO.

3

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 silt to sandy silt / till deposits should be calculated from the following expressions:

$$R \text{ (silt to sand)} = 1.5\ D\ Kp\ L^2 \gamma$$

where			
	R	= Ultimate Load to be restrained	kN
	D	= Diameter of concrete filled hole	m
	Kp	= Passive resistance in the deposit	3.0
	L	= Embedment Depth of the pile	m
	γ	= Unit weight of the soil - use 21 kN/m^3 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:

Thickness of lagging	Maximum Spacing of Soldier Piles
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 silt to sandy silt / till deposits, can be estimated using an adhesion value 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 the silt to sandy silt or clayey silt deposits should be designed for allowable bearing pressures of 200 kPa (8.0 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.

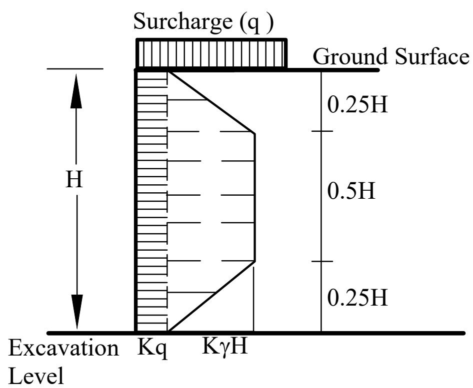
6. General Shoring Notes

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.

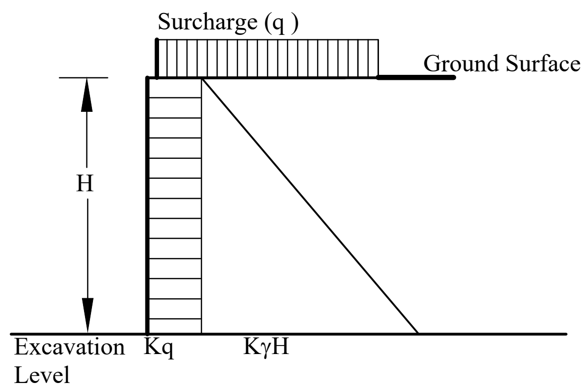
TEMPORARY SHORING

Laterial Pressure

I. Multiple Level Support



II. Single Level Support



Lateral Pressure P= K(γH + q)

where	H = Height of Shoring	m
	γ = Unit Weight of Retained Soil	21.0 kN/ m^3
	q = Surcharge	kPa
	K = Earth Pressure Coefficient	

If moderate ground and shoring movements are permissible then:
K = Ka = Active Earth Pressure Coeffiecient = 0.25

If there are building foundations within a distance of 0.5 H behind the shoring then:
K = Ko = Earth Pressure at rest = 0.4

If there are building foundations within a distance of between 0.5 H and H behind the shoring then:
K = 0.5 (Ka + Ko) = 0.33

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

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TITLE: Temporary Shoring Design

DRAWING NO.: A1