Proposed Residential Development

CreateTO

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Geotechnical Investigation

Project Location:
15 Denison Avenue, Toronto, Ontario

Project Number:
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Table of Contents

1. Introduction

This report presents the findings of a geotechnical investigation conducted for the proposed residential development at 15 Denison Avenue in the City of Toronto, Ontario. The work was authorized by Ms. Tracey Smith of CreateTO.

It is noted that the final development plans, including building orientation and location, for the site are not known at the time of the investigation. However, it is understood that the proposed development will be an eight (8) storey structure with two (2) levels of underground parking occupying the entire property.

The purpose of the geotechnical investigation was to determine the subsurface soil and groundwater conditions at the site by putting down limited sampled boreholes and, based on an assessment of the factual borehole data, to provide geotechnical engineering guidelines for the design and construction of proposed development.

Our Terms of Reference also included a Phase I and II Environmental Site Assessment (ESA) and a hydrogeological study. The results of the Phase I and II ESA and hydrogeological study will be presented under separate covers.

The comments and recommendations given in this report are based on the assumption the above-described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these changes. The result of this review may be a modification of our recommendations or the requirement of additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2. Site Description

The site is located approximately 300 m northeast of the intersection of Bathurst Street and Queen Street West in the City of Toronto, Ontario. The municipal address of the site is 15 Denison Avenue, Toronto, Ontario.

The site is rectangular in shape and is approximately 2,800 m^2 (0.28 hectares) in size. It is currently a paved parking lot operated by Toronto Parking Authority (TPA). Based on the elevations of the boreholes, the site is relatively flat with elevations ranging between 95.15 and 95.58 m. The site is bounded by Augusta Avenue to the east, Wolseley Street to the north, Denison Avenue to the west and private laneway to the south.

3. Fieldwork

The fieldwork was carried out between December 6 and 11, 2021. Prior to drilling, the borehole locations were cleared of underground utilities by Ontario One Call contractors and a private utility locater. Five (5) boreholes were advanced to depths of about 9.8 to 19.8 m below existing grade at the approximate location shown on the attached Borehole Location Plan (Drawing No. 1).

The boreholes were advanced using continuous flight solid or hollow stem augering equipment owned and operated by a specialist drilling contractor. In the boreholes, soil samples were recovered using conventional split spoon equipment and standard penetration test methods. To confirm bedrock and to determine its quality, Borehole 5 was extended 2.6 m into the bedrock by coring in NQ size using diamond drilling equipment.

Water levels were observed in the boreholes during the course of the fieldwork and in monitoring wells installed in all boreholes to establish the short-term stabilized groundwater level at the site. The monitoring wells were installed in accordance with the Ontario Water Resources Act, R.R.O. 1990, Ontario Regulation (O. Reg.) 903 – Amended to O. Reg. 128/03.

The fieldwork was supervised by EXP geotechnical staff who monitored the drilling operations and logged the borings. The split spoon samples and recovered rock cores were transported to our laboratory for detailed examination.

The location and ground surface elevation of the boreholes were determined in the field by EXP Services Inc. The ground surface elevation at each borehole location was referred to the following City of Toronto benchmark (B.M.).

B.M.: East side of Augusta Avenue, opposite Carr Avenue. Monument on multi-storey apartment building. 0.3 m east from northwest corner. 0.6 m above grade.

Elevation: 95.938 m (geodetic).

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4. Subsurface Conditions

4.1 Soil and Bedrock

The detailed soil and bedrock profiles encountered in each borehole is shown on the attached borehole logs (Drawing No. 2 to 6). It should be noted the boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The “Notes on Sample Descriptions” preceding the borehole logs form an integral part of and should be read in conjunction with this report.

The stratigraphy of the site, as revealed in the boreholes, generally comprised surficial pavement structure underlain by fill followed by native deposits of sandy silt till, clayey silt till and silt till overlying shale bedrock.

A brief description of the stratigraphy, in order of depth, follows:

Pavement Structure

Pavement structure, comprising 150 to 165 mm of asphaltic concrete over of 350 to 535 mm granular material, was encountered at the surface in all boreholes.

Fill

Fill was encountered below the pavement structure in all boreholes and generally consisted of brown silty sand to sandy silt with trace to some gravel. Brick fragments were noted within the fill in Borehole 2. Moisture content ranged from 5 to 10%, indicating a moist condition. The fill stratum extended to depths of about 1.4 to 2.1 m below existing grade.

Sandy Silt Till

A sandy silt till deposit was encountered below the fill in Boreholes 1 and 3. This deposit extended to depths of about 2.1 to 2.9 m below existing grade. The sandy silt till was brown in colour and contained trace clay and gravel. The moist sandy silt till existed in a compact state of compactness with moisture contents ranging between 9 and 10%.

Clayey Silt Till

Below the fill in Boreholes 2, 4 and 5 and the sandy silt till in Boreholes 1 and 3, a clayey silt till deposit was encountered and extended to depths of about 5.9 to 6.4 m below existing grade. The brown/grey to grey clayey silt till contained trace gravel and was stiff to very stiff in consistency. Moisture contents ranged from 12 to 17%, indicating moist to very moist conditions.

Silt Till

A silt till deposit was encountered at depths of about 16.6 to 20.7 m in Boreholes 1, 3 and 4, extending to depths of about 14.9 to 15.2 m below existing grade in Boreholes 3 to 5 and to the termination depth of boreholes at about 9.8 to 13.8 m below existing grade in Boreholes 1 and 2. This grey silt till contained trace to some clay and sand and trace gravel. This deposit existed in a dense to very dense state of compactness. Numerous sand seams/layers were noted in this silt till deposit. Moisture contents ranged from 9 to 13%, indicating moist conditions.

Shale Bedrock

In Boreholes 3 to 5, weathered shale bedrock was encountered at about 14.9 to 15.2 m below the existing ground surface, corresponding to approximately Elevations 80.2 to 80.5 m, as identified by increased drilling resistance and the visual classification of the recovered samples. It should be noted that the upper zone of the bedrock is generally completely to highly weathered. The distinction between completely weathered shale and the overlying strata, particularly if the latter contains abundant shale fragments, is not always clear and consequently, some of the soils resting on the surface of the bedrock might be very weak or completely weathered rock. As such, the contact elevations should not be interpreted as exact planes of bedrock since the auger will frequently penetrate some distance into the weathered rock before noticeable resistance is encountered.

Coring of rock was carried out in Borehole 5 to confirm and determine the quality of the bedrock for this investigation. The detailed findings from the rock samples are presented in the rock log. An explanation is given on the “Explanation of Terms Used in the Bedrock Core Log” sheet preceding the rock core log and should be read in conjunction with this report.

Based on the examination of the recovered rock core samples, the shale bedrock generally consists of thinly bedded or laminated grey shale with limestone interbeds and is completely weathered to highly weathered in the upper 3 m, becoming moderately to slightly weathered with depth. The Rock Quality Designation (RQD), a rock quality indicator, is defined as the sum of core lengths of 100 mm or greater divided by the total length of the drill run. The RQD for the rock cores ranged from about 39 to 52%, indicating very poor to poor quality.

Based on the knowledge of the site area, the bedrock at this site is the Georgian Bay Formation which consists of bluish and grey shale with interbeds of siltstone, limestone and dolostone. Typically, the hard layers comprise about 15 to 20 percent of the unit. The hard layers are usually less than about 100 to 150 mm thick but some are much thicker. The thicker layers have been observed to be as much as 750 to 900 mm at other sites. The layers are actually lenses and they can vary significantly in thickness over short distances.

Stress relief features such as folds and faults are common in the Georgian Bay Formation. In these features the rock is heavily fractured and sheared. It can also contain layers of shale rubble and clay. Due to the fracturing, these features may also be groundwater conduits, which could result in excessive water flow into excavation. Weathering is much deeper than the surrounding rock in sound unweathered bedrock overlying fractured and weathered bedrock. The stress relief features are usually in the order of 4 to 6 m wide, but in depth can vary from 4 to 5 m to in excess of 10 m.

4.2 Groundwater

Groundwater conditions were observed in the borehole during the course of the fieldwork. Short-term groundwater measurement is included in the attached borehole log.

Upon completion of drilling, groundwater level of 7.93 m below existing grade was observed in Boreholes 204 and 205. The groundwater was unable to be measured in Boreholes 201 to 203 due to the use of water during drilling operation. The subsequent short-term groundwater level readings are presented in Table 1 below.

Table 1: Short-Term Groundwater Levels in Borehole Locations

Borehole No.	Approximate Ground Surface Elevation (m)	Groundwater Level below Existing Grade / Elevation in Groundwater Monitoring Well on January 11, 2020 (m)	Groundwater Level below Existing Grade / Elevation in Groundwater Monitoring Well on January 13, 2020 (m)	Groundwater Level below Existing Grade / Elevation in Groundwater Monitoring Well on January 26, 2020 (m)
BH 1	95.33	Dry	8.55 / ~86.8	8.47 / ~86.9
BH 2	95.58	8.93 / ~86.7	7.76 / ~87.8	7.56 / ~88.0
BH 3	95.76	6.99 / ~88.8	6.54 / ~89.2	6.17 / ~89.6
BH 4	95.58	6.65 / ~88.9	6.65 / ~88.9	6.75 / ~88.8
BH 5	95.15	7.68 / ~87.5	6.63 / ~88.5	4.75 / ~90.4

The groundwater elevations reflect conditions at the time of the investigation. Seasonal fluctuation of the groundwater levels at the site should be anticipated.

Reference should be made to the hydrogeological study for details of the groundwater conditions at this site.

5. Engineering Discussion and Recommendations

5.1 General

A geotechnical investigation was conducted for the proposed residential development at 15 Denison Avenue in the City of Toronto, Ontario. It is noted that the final development plans, including building orientation and location, for the site is not known at the time of the investigation. However, it is understood that the proposed development will be an eight (8) storey structure with two (2) levels of underground parking occupying the entire property.

The following subsections provide geotechnical engineering guidelines pertinent to the design and construction of the proposed development.

5.2 Foundations Considerations

Based on the results of the limited boreholes drilled at the site, it is considered the site will be suitable for the construction of the proposed development.

For the proposed structure with two (2) levels of underground parking, two (2) foundation systems may be considered, namely:

1. Conventional Footings
2. Cast-in-Place Concrete Caissons

The preferred means of supporting the structure will be dictated by economics, design and construction constraint.

5.2.1 Conventional Spread and Strip Footings

For the proposed structure with two (2) levels of underground parking, it is anticipated the basement slab will be at a depth of about 6 to 7 m below the existing ground surface. At that depth, dense to very dense silt till will be encountered.

It is considered the proposed structure may be supported on a conventional spread and strip footings founded on the competent native dense to very dense silt till. For preliminary design purposes, footings founded on the very dense silt till at or below 7 m below existing grade may be designed for a geotechnical resistance of 500 kPa at Serviceability Limit States (S.L.S.), subject to inspection during construction. The factored geotechnical resistance at Ultimate Limit States (U.L.S.) is 750 kPa. Table 2 below provides a summary of the highest founding depth where the recommended geotechnical resistance of 500 kPa (S.L.S.) can be applied for footings.

Table 2: Summary of the Highest Founding Elevations where the Recommended Geotechnical Reaction of 500 kPa (S.L.S.) can be applied for Footings

Borehole No.	Approximate Ground Surface Elevation (m)	Highest Founding Elevation (Depth below ex. Grade) (m)
BH 1	95.33	~88.8 (6.5)
BH 2	95.58	~88.6 (7.0)
BH 3	95.76	~88.8 (7.0)
BH 4	95.58	~88.6 (7.0)
BH 5	95.15	~88.2 (7.0)

Prior to placement of concrete, all footing bases should be inspected by geotechnical personnel from EXP Services Inc. to verify the competency of the founding material.

5.2.2 Cast-in-Place Concrete Caissons

Should a higher bearing capacity be required, the structure may also be supported on a deep foundation founded on the underlying sound shale bedrock.

It is recommended the proposed structure be supported on drilled caissons founded in the underlying sound shale bedrock below all weathered zones and loose rock. Based on the information obtained from the rock coring, the upper 3 m of the shale bedrock is completely to highly weathered. For preliminary design purposes, caissons on the sound shale bedrock may be designed for a geotechnical resistance of 7,200 kPa at Ultimate Limit States (U.L.S.), subject to inspection during construction. The geotechnical resistance at Serviceability Limit States (S.L.S.) does not govern as bedrock is considered to be 'unyielding' soil. It is recommended additional boreholes, including rock coring, be carried out to further determine the quality of the bedrock.

Since down hole cleaning of the caisson base is not permitted, the caisson can be cleaned by the augers at the initial phase. The final cleaning of the caisson bases can then be auger cleaned by mixing the loose materials at the base of the caissons with 0.3 to 0.5 m thick concrete. The mixture should then be removed. During the installation of caissons, a temporary steel liner will have to be installed to prevent the caving of the drilled hole and to seal off any water which may be present in the water bearing silty sand above the founding levels. A positive head of concrete inside the liner with respect to any exterior groundwater levels must be maintained during withdrawal of the liner. A 150 mm slump concrete is recommended for use to prevent the concrete from having a honeycombed structure and to avoid bridging in the liner upon its withdrawal. If there are significant amount of water in the caisson base and cannot be bailed out, the concrete should be placed using tremie method.

As per the Ontario Building Code, the installation operation for each caisson must be monitored on a continuous (full-time) basis by geotechnical personnel to verify the founding soil, foundation elevations and alignment.

5.2.3 Foundations General

Footings/caissons which are to be placed at different elevations should be located such that the higher footing/caisson is set below a line drawn up at 10 horizontal to 7 vertical from the near edge of the lower footing/caisson.

All footings/grade beams exposed to seasonal freezing conditions should be protected from frost action by at least 1.2 m of soil cover or equivalent insulation, depending on the final design requirements. However, for footings below 2 unheated levels of basement, unmonitored experience in recent years indicates shallow footing depths of 1.0 m for interior columns and 0.6 m for walls have been successful. During winter months, the exposed footings should be covered with thermal blankets to prevent frost penetration below the footings which may result in unacceptable movements (i.e. frost heave). Adjacent to air shafts and entrance and exit doors, a footing depth of 1.2 m below floor surface level is required.

The total and differential settlements of well designed and constructed footings placed in accordance with the above recommendations are expected to be less than 25 mm and 20 mm, respectively.

It should be noted the recommended bearing value has been calculated by EXP from the borehole information for the design stage only. The investigation and comments are necessarily ongoing as new information on underground conditions becomes available. For example, it should be appreciated modification to bearing levels may be required if unforeseen subsurface conditions are encountered or if final design decisions differ from those assumed in this report. For this reason, this office should be retained to review final foundation drawings and to provide field inspections during the construction stage.

5.3 Shoring Requirements

Shoring will be required along each wall of the excavation to limit the horizontal and vertical movements of adjacent properties, buried utilities and roadways. A shoring system consisting of tied-back soldier piles and lagging is expected to provide suitable support in areas where some movements are acceptable. In areas where movement are to be minimized or sensitive, a continuous caisson wall supported by tiebacks may be required to provide lateral support.

The shoring systems should be designed in accordance with the latest edition of the Canadian Foundation Engineering Manual (CFEM). Based on the manual, the following earth-pressure coefficients are recommended.

0.25 Where minor movements can be tolerated.

0.35 Where utilities, roads, sidewalks must be protected from significant movement or where vibration from traffic is a factor.

0.45 Where movements are to be minimized such as near adjacent building footings or movement sensitive services (i.e. gas and watermains).

Approximate Soil Unit Weight = 22.0 kN/m^3 (soil)

Unit Weight of Groundwater = 9.8 kN/m^3

Bond resistance in clayey silt till and silt till
= 50 kPa (higher values can be obtained if re-groutable anchors are used)

The shoring system should be designed by a specialist shoring contractor. All soldier pile and tieback holes should be temporary cased to minimize the risk of caving. During winter months, the shoring should be covered with thermal blankets to prevent frost penetration behind the shoring system which may result in unacceptable movements.

The recommended design parameters should be confirmed by sufficient full scale pull-out tests ("performance test") in accordance with the Post-Tensioning Institute (PTI) guidelines. The design for the production anchors should then be modified based on the test results, where necessary. All remaining anchors must be installed using similar procedures and proof tested to 1.33 times the design load.

EXP should be retained to review the shoring design, to monitor installation and testing of the system, and to monitor the shoring movements during all phases of the excavation. Inclinometers should be installed at locations where buildings or sensitive services lie close to the excavation. Careful monitoring is needed in any shored excavation, especially when buildings are located in close proximity. This is necessary not only to anticipate when and if additional support is needed, but also to provide data to meet claims from adjacent property owners. In this regard, it is essential detailed pre-condition surveys be conducted on adjacent buildings.

5.4 Excavation and Groundwater Control

Following completion of the shoring installation, excavation for basement and foundation construction may proceed. Excavation through the overburden soils should be relatively straightforward using conventional equipment. Excavation must be carried out in accordance with the Occupational Health and Safety Act and local regulations.

It should be noted obstructions and cobbles and boulders may be present within the fill and till deposits. Consequently, provisions should be made in the contract documents to cover any delays caused by obstructions and cobbles and boulders.

Based on the groundwater conditions encountered, groundwater seepage should be anticipated for construction of two (2) levels of underground parking extending to a depth of about 6 to 7 m below existing grade. However, it is anticipated that the rate of seepage through the till deposits would be slow. As such, it is in our opinion that minor seepage entering the excavation from water perched in the fill and pervious seams and layers in the native deposits during construction should be possible to control using conventional construction dewatering techniques, i.e. pumping from localized sumps.

It should be noted that any temporary construction dewatering that extracts more than 400,000 L per day would be subjected to a Permit to Take Water (PTTW), as regulated by the Ministry of the Environment, Conservation and Parks (MECP). If the estimated rates will be more than 50,000 L per day but less than 400,000 L per day, the water taking can be registered under the Environmental Activity and Sector Registry (EASR) as per MOECC’s new regulatory requirements.

For short and long-term groundwater control requirements, a Hydrogeological Study has been carried out to determine if a PTTW is required from the MECP and as part of the submission to the City of Toronto.

5.5 Backfill Consideration

Backfill used to satisfy underfloor slab requirements, in footing and service trenches, etc., should be compactible fill, i.e. inorganic soil with its moisture content close to its optimum value determined in the standard Proctor maximum dry density test. The excavated materials will consist of fill, sandy silt till, clayey silt till and silt till. Fill containing organics inclusions and otherwise deleterious materials is considered not suitable for backfilling purposes. The native soils are suitable for backfilling purposes; however, some moisture adjustments (i.e. drying) will be required prior to be suitable for reuse as backfill material.

Any shortfall of suitable on-site excavated material can be made up with imported granular material, OPSS Granular ‘B’ or equivalent. The backfill should be placed in lifts not more than 300 mm thick in the loose state with each lift being compacted to 98% standard Proctor maximum dry density (SPMDD) before subsequent lifts are placed. The degree of compaction achieved in the field should be checked by in-place density tests.

The overburden soils are not free draining and therefore should not be used where this characteristic is required or in confined areas where smaller compaction equipment is required. Imported granular material conforming to OPSS Granular ‘B’ would also be suitable for these purposes.

5.6 Floor Slab Construction and Permanent Drainage

It is anticipated the lowest basement floor slab can be constructed as a slab-on-grade on the native soils. Prior to slab-on-grade construction, the exposed subgrade surface should be compacted and proofrolled with a heavy roller and examined by geotechnical personnel. Any soft areas identified during the proofrolling operation should be subexcavated and replaced with approved material compacted in the manner described in the "Backfill Considerations" subsection of the report.

A moisture barrier, consisting of a 200 mm thick layer of 19 mm clear crushed stone should be placed directly under the floor slab.

Perimeter drainage is required to remove any water that may be accumulated at the exterior foundation walls. To prevent build-up of water adjacent to the basement walls, it would be prudent to incorporate an exterior vertical drainage sheet attached to the backside of the basement wall connected to frost free outlets inside the building. The exterior vertical drainage sheet should consist of SITEDRAIN HQ 240 or equivalent covering the entire basement wall in order to reduce the risk of water penetration. The wall drain panels should be outletted through the basement wall into the basement. A solid pipe should be installed to within 1 m of the exterior wall to collect seepage for the wall drains.

In addition, installation of an under-floor drain system is also recommended below the basement slab. For preliminary guidance, the underfloor drain system should consist of a 200 mm thick layer of clear stone, with 100 mm diameter perforated drain pipes installed at the base of the drainage stone, at 6 m intervals. The final spacing of the underground drains should be determined by the hydrological study. The pipes and the stone must be completely wrapped in a non-woven geotextile. These drain pipes must be provided with a frost-free positive discharge (i.e. sump pits). Adequate clean-out ports should be installed for each line of drainage pipes to faciliate future cleaning of the pipes.

The perimeter and sub-floor drainage systems should be independent of any stormwater piping, such as rainwater leaders. Backflow prevention should be provided between the sumps and the drain headers.

The City of Toronto requires a detailed Hydrogeological Study be carried out for each site to determine the short-term (during construction) and long-term (post construction) flow rates. The hydrogeological report will be reviewed by the City of Toronto to determine if the groundwater is allowed to be discharged into their sewer system based on the quantity and quality of the water. If the groundwater collected from perimeter and underfloor drainage system are not allowed to be discharged into the City sewers, the basement walls and floor will need to be designed as a watertight structure. In this case, the walls and floor must be designed to resist the hydrostatic pressures exerted by the recorded groundwater levels.

5.7 Earth Pressure on Subsurface Walls

The lateral earth pressure acting on basement walls may be calculated from the following equation:

$$p = k(\gamma h + q)$$

where:

- p = the pressure in kPa acting against any subsurface wall at depth, h, below the ground surface;
- k = the earth pressure coefficient considered to be appropriate for the subsurface walls, for this case, 0.4;
- γ = the bulk unit weight of the retained soil; use 22 kN/m^3;
- h = the depth in m below the ground surface at which the pressure, p, is to be computed; and
- q = the value of any adjacent surcharge in kPa which may be acting close to the wall.

The above expression assumes an effective perimeter drainage system will be incorporated to prevent the build-up of hydrostatic pressure behind the subsurface wall. The subsurface walls should be properly waterproofed. If both the subsurface walls and floor are tanked, they should be designed to resist full hydrostatic pressures and uplift.

If water is retained, submerged unit weight can be used for the retained soil below the groundwater table and full hydrostatic pressure should be added to the above equation. Accordingly, the lateral earth pressures acting on basement walls may be calculated from the following expression:

$$p = K(\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2$$

Where:

- p = lateral earth pressure in kPa acting at depth h
- K = earth pressure coefficient, assumed to be (0.4) for vertical walls and horizontal backfill
- γ = unit weight of soil, a value of 22 kN/m^3 may be assumed
- h_1 = depth in meters above the water table
- γ' = Effective unit weight of soil, a value of 12 kN/ m^3 may be assumed
- γ_w = unit weight of water (9.8 kN/ m^3)
- h_2 = depth in metres below the water table
- q = equivalent value of surcharge on the ground surface in kPa

5.8 Earthquake Consideration

The recommendations for the geotechnical aspects to determine the earthquake loading are presented below.

6.8.1 Subsoil Conditions

The subsoil information at this site has been examined in relation to Section 4.1.8.4 of OBC 2012. The subsoil consisted of fill underlain by native sandy silt till, clayey silt till and silt till overlying shale bedrock. The proposed structure will be supported on conventional footings founded on the dense to very dense silt till or caissons founded in sound shale bedrock.

There have been no shear wave velocity measurements carried out at this site.

6.8.2 Depth of Boreholes

Table 4.1.8.4.A Site Classification for Seismic Site Response in OBC 2012 indicated that to determine the site classification, the average properties in the top 30 m are to be used. The boreholes were advanced to depths of 9.8 to 19.8 m below existing grade. Shale bedrock was encountered at depths of about 14.9 to 15.2 m below existing grade.

6.8.3 Site Classification

Based on the known soil conditions, the Site Class for this site should be “C” as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012.

6. General Comments

A geotechnical engineer should be retained for a general review of the final design and specifications to verify the recommendations in this report address all the relevant geotechnical parameters regarding the design and construction of the proposed development.

The comments given in this report are intended only for the guidance of design and structural engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations as well as their own interpretations of the factual borehole results so that they may draw their own conclusions as to how the subsurface conditions may affect them.

More specific information with respect to the conditions between samples or the lateral and vertical extent of materials may become apparent during excavation operations. The interpretation of the borehole information must, therefore, be validated during excavation operations. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent; should this occur, a geotechnical engineer should be contacted to assess the situation and additional testing and reporting may be required. EXP has qualified personnel to provide assistance in regard to future geotechnical issues related to this property.

We trust this report is satisfactory for your purposes. Should you have any questions or comments, please do not hesitate to contact this office.

Yours truly,

EXP Services Inc.

Leo Chui, P. Eng.
Project Manager
Geotechnical Services

Peter Chan, P. Eng.
Vice President, Central Ontario
Geotechnical Services

Drawings

Borehole Location Plans
Borehole Logs

NOTES:

1. THE BOUNDARIES AND SOIL TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR
2. SOIL SAMPLES WILL BE RETAINED IN STORAGE FOR 1 MONTH AND THEN DESTROYED UNLESS CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. BOREHOLE ELEVATIONS SHOULD NOT BE USED FOR BUILDING GRADES.

LEGEND:

BOREHOLE LOCATION

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PROJECT TITLE AND LOCATION:	DRAWING TITLE:	PROJECT #: GTR–21028382–A0	DWN.: LC
GEOTECHNICAL INVESTIGATION PROPOSED RESIDENTIAL DEVELOPMENT 15 DENISON AVENUE TORONTO, ONTARIO	BOREHOLE LOCATION PLAN	SCALE: 1:650	CHKD.: PC
		DATE: JANUARY 2022	DWG. No.: 1

Notes on Sample Descriptions

1. All sample descriptions included in this report follow the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), as outlined in the Canadian Foundation Engineering Manual. Note, however, that behavioral properties (i.e. plasticity, permeability) take precedence over particle gradation when classifying soil. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.

UNIFIED SOIL CLASSIFICATION

CLAY (PLASTIC) TO	FINE	MEDIUM	CRS.	FINE	COARSE
SILT (NONPLASTIC)	SAND			GRAVEL	

0.002 0.006 0.02 0.06 0.2 0.6 2.0 6.0 20 60 200

EQUIVALENT GRAIN DIAMETER IN MILLIMETRES

ISSMFE SOIL CLASSIFICATION

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Notes On Soil Descriptions

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay and Silt	<0.060 mm	"trace" (e.g. Trace sand)	1% to 10%
Sand	0.060 to 2.0 mm	"some" (e.g. Some sand)	10% to 20%
Gravel	2.0 to 75 mm	adjective (e.g. sandy, silty)	20% to 35%
Cobbles	75 to 200 mm	"and" (e.g. and sand)	35% to 50%
Boulders	>200 mm		

The compactness of Cohesionless soils and the consistency of the cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil		
Compactness	**Standard Penetration Resistance "N" Blows / 0.3 m**	**Consistency**	**Undrained Shear Strength (kPa)**	**Standard Penetration Resistance "N" Blows / 0.3 m**
Very Loose	0 to 4	Very soft	<12	<2
Loose	4 to 10	Soft	12 to 25	2 to 4
Compact	10 to 30	Firm	25 to 50	4 to 8
Dense	30 to 50	Stiff	50 to 100	8 to 15
Very Dense	Over 50	Very Stiff	100 to 200	15 to 30
		Hard	>200	>30

5. **ROCK CORING**

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundless of the rock mass. It is obtained from the rock cores by summing the length of the core covered, counting only those pieces of sound core that are 100 mm or more length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

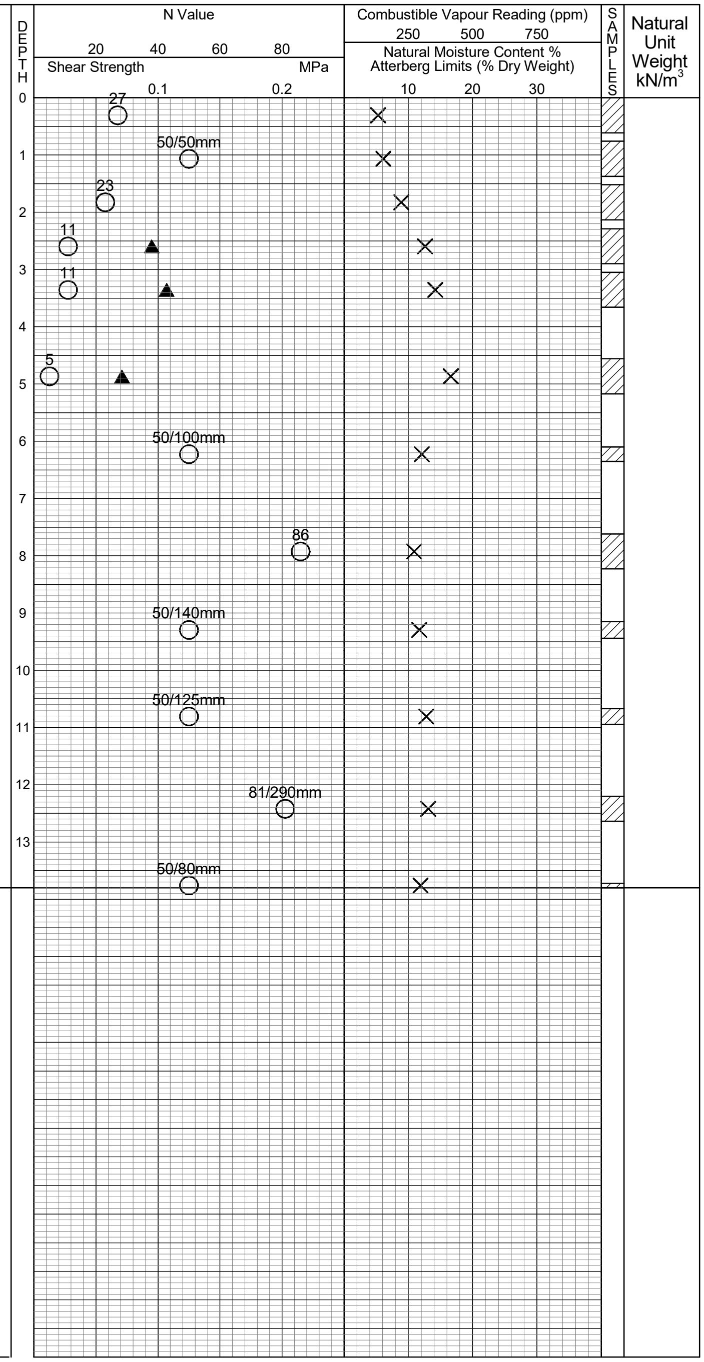
Log of Borehole 1

Project No.	GTR-21018382-A0	Drawing No.	2
Project:	Proposed Residential Development	Sheet No.	1 of 1
Location:	15 Denison Avenue, Toronto, Ontario		

Date Drilled:	December 6, 2021
Drill Type:	M5T Truck Mount
Datum:	Geodetic

Legend:
- Auger Sample
- SPT (N) Value
- Dynamic Cone Test
- Shelby Tube
- Field Vane Test
- Combustible Vapour Reading
- Natural Moisture
- Plastic and Liquid Limit
- Undrained Triaxial at % Strain at Failure
- Penetrometer

ELEV. m	Soil Description
95.33	
95.2	**PAVEMENT STRUCTURE** - 150 mm asphaltic concrete
94.7	- over 450 mm granular material
93.6	**FILL** - silty sand to sandy silt, trace to some gravel; brown to grey/brown, moist
93.2	**SANDY SILT TILL** - trace clay, trace gravel; brown, moist compact
	CLAYEY SILT TILL - trace gravel; brown, moist, stiff - becoming grey - becoming very moist
89.4	**SILT TILL** - trace to some clay, trace sand, trace gravel; grey, moist, very dense
81.5	**END OF BOREHOLE**



NOTES:
1. Groundwater monitoring well installed to 9.15 m depth.

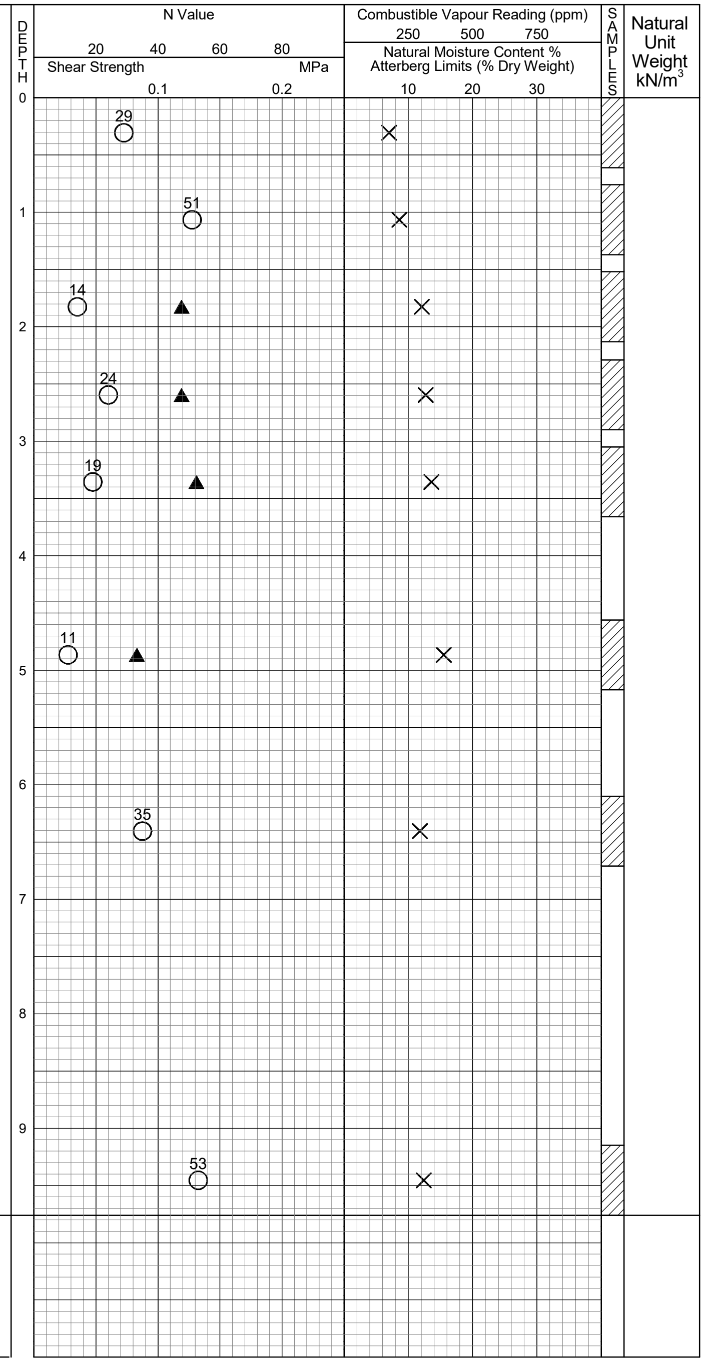
Time	Water Level (m)	Depth to Cave (m)
On completion	N/A	Well
January 11, 2022	Dry	Well
January 13, 2022	8.55	Well
January 26, 2022	8.47	Well

Log of Borehole 2

Project No.	GTR-21018382-A0	Drawing No.	3
Project:	Proposed Residential Development	Sheet No.	1 of 1
Location:	15 Denison Avenue, Toronto, Ontario		
Date Drilled:	December 11, 2021		
Drill Type:	M5T Truck Mount		
Datum:	Geodetic		

Legend		Legend	
Auger Sample	⊠	Combustible Vapour Reading	□
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	—	Plastic and Liquid Limit	├—○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

Soil Description	ELEV. m
	95.58
PAVEMENT STRUCTURE - 150 mm asphaltic concrete	95.4
- over 450 mm granular material	95.0
FILL - sandy silt, trace to some gravel, brick fragments; brown to brown grey, moist	94.2
CLAYEY SILT TILL - trace gravel; brown, moist, stiff - becoming grey - becoming very moist	89.7
SILT TILL - trace to some clay, trace sand, trace gravel; grey, moist, very dense	85.8
END OF BOREHOLE **NOTES:** 1. Groundwater monitoring well installed to 9.15 m depth.	



Time	Water Level (m)	Depth to Cave (m)
On completion	N/A	Well
January 11, 2022	8.93	Well
January 13, 2022	7.76	Well
January 26, 2022	7.56	Well

exp.

Log of Borehole 3

Project No. GTR-21018382-A0 — Drawing No. 4

Project: Proposed Residential Development — Sheet No. 1 of 1

Location: 15 Denison Avenue, Toronto, Ontario

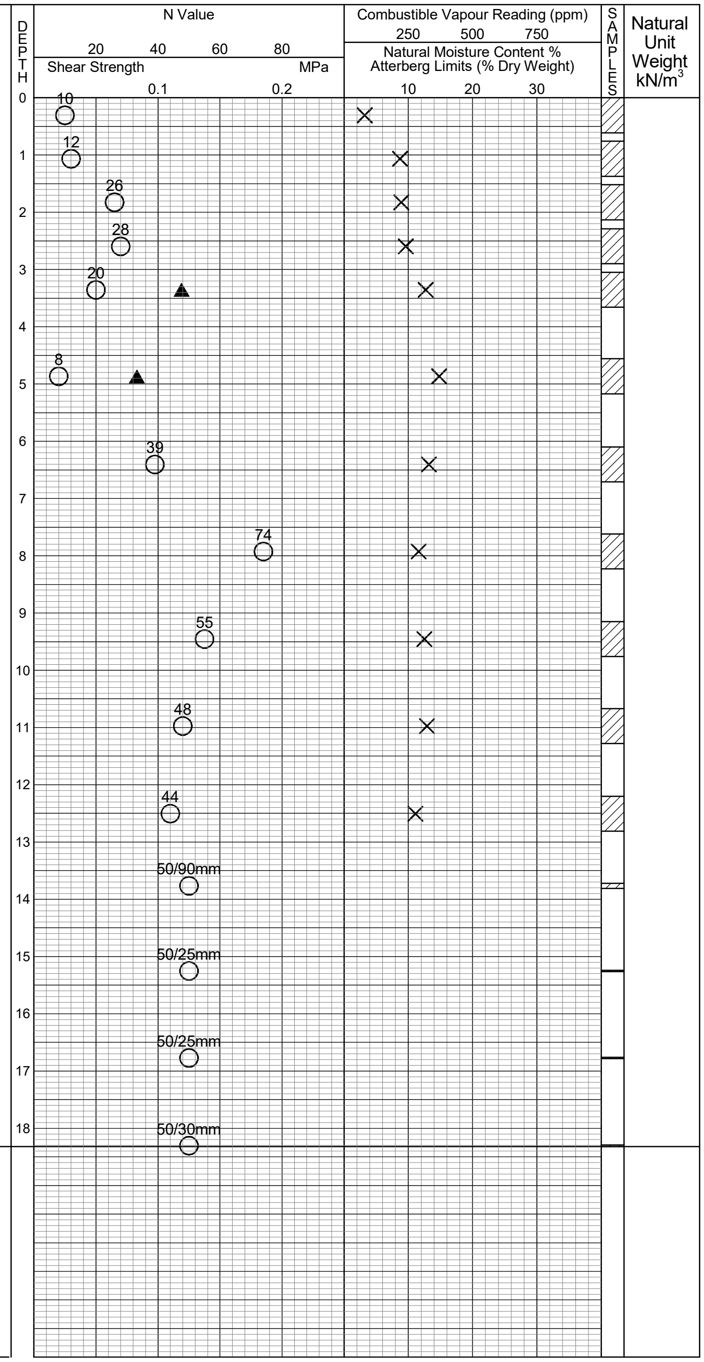
Date Drilled: December 6-7, 2021

Drill Type: M5T Truck Mount

Datum: Geodetic

Legend		Legend	
Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	×
Dynamic Cone Test	——	Plastic and Liquid Limit	├──○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

Soil Description	ELEV. m
	95.76
PAVEMENT STRUCTURE - 165 mm asphaltic concrete	95.6
- over 435 mm granular material	95.2
FILL - sandy silt, trace clay, trace gravel; brown, moist	94.4
SANDY SILT TILL - trace clay, trace gravel; brown, moist compact	92.9
CLAYEY SILT TILL - trace gravel; brown/grey, moist, stiff to very stiff - becoming grey, very moist	89.8
SILT TILL - trace to some clay, trace to some sand, trace gravel, numerous sand seams; grey, moist, dense to very dense	80.5
WEATHERED SHALE - grey, moist	77.4
END OF BOREHOLE	



NOTES:
1. Groundwater monitoring well installed to 9.15 m depth.

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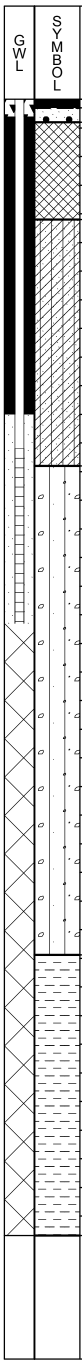
Time	Water Level (m)	Depth to Cave (m)
On completion	N/A	Well
January 11, 2022	6.99	Well
January 13, 2022	6.54	Well
January 26, 2022	6.17	Well

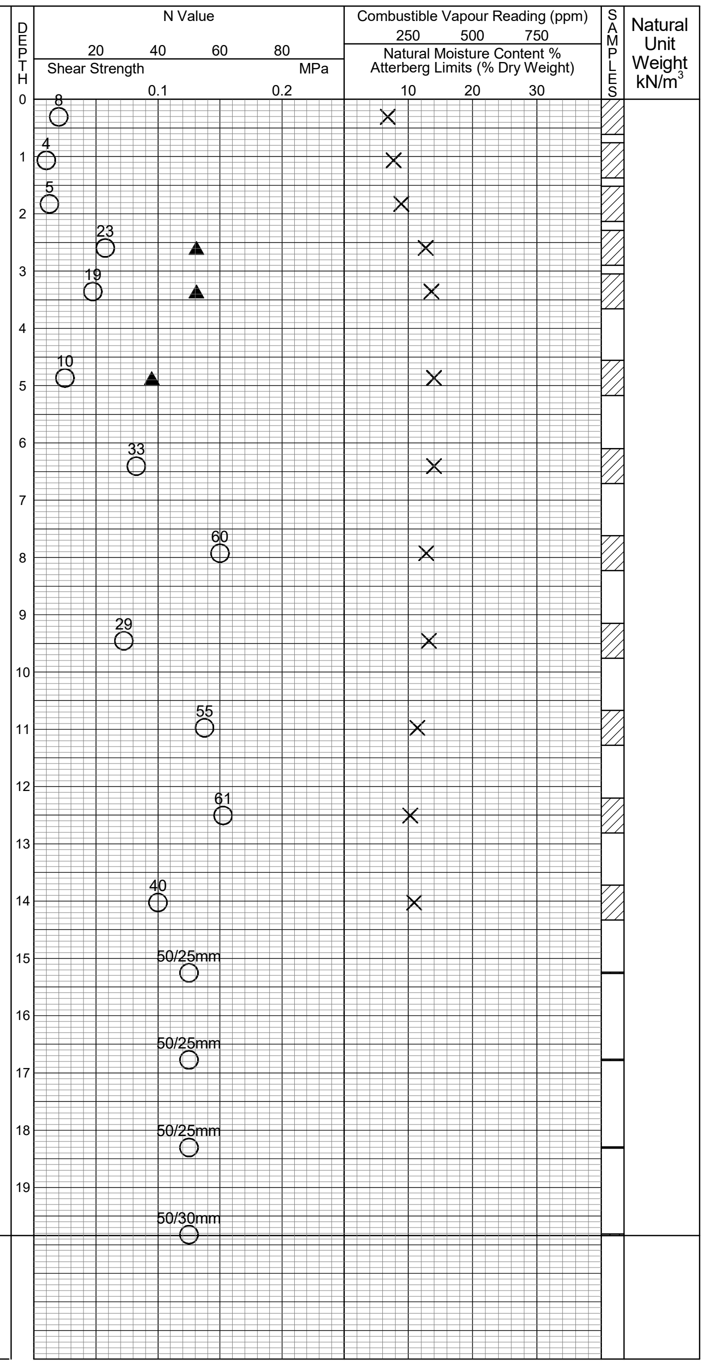
Log of Borehole 4

Project No.	GTR-21018382-A0	Drawing No.	5
Project:	Proposed Residential Development	Sheet No.	1 of 1
Location:	15 Denison Avenue, Toronto, Ontario		
Date Drilled:	December 7, 2021		
Drill Type:	M5T Truck Mount		
Datum:	Geodetic		

Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test
Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer

GWL	SYMBOL	Soil Description	ELEV. m
			95.58
		PAVEMENT STRUCTURE - 150 mm asphaltic concrete	95.4
		- over 350 mm granular material	95.2
		FILL - silty sand to sandy silt, trace to some gravel; brown, moist	93.5
		CLAYEY SILT TILL - trace gravel; brown/grey, moist, stiff to very stiff - becoming grey, very moist	89.2
		SILT TILL - trace to some clay, trace to some sand, trace gravel, numerous sand seams; grey, moist, dense to very dense	80.6
		WEATHERED SHALE - grey, moist	75.7
		END OF BOREHOLE **NOTES:** 1. Groundwater monitoring well installed to 9.15 m depth.	





Time	Water Level (m)	Depth to Cave (m)
On completion	N/A	Well
January 11, 2022	6.65	Well
January 13, 2022	6.65	Well
January 26, 2022	6.75	Well

exp.

Log of Borehole 5

Project No.	GTR-21018382-A0	Drawing No.	6
Project:	Proposed Residential Development	Sheet No.	1 of 1
Location:	15 Denison Avenue, Toronto, Ontario		
Date Drilled:	December 9, 2021		
Drill Type:	M5T Truck Mount		
Datum:	Geodetic		

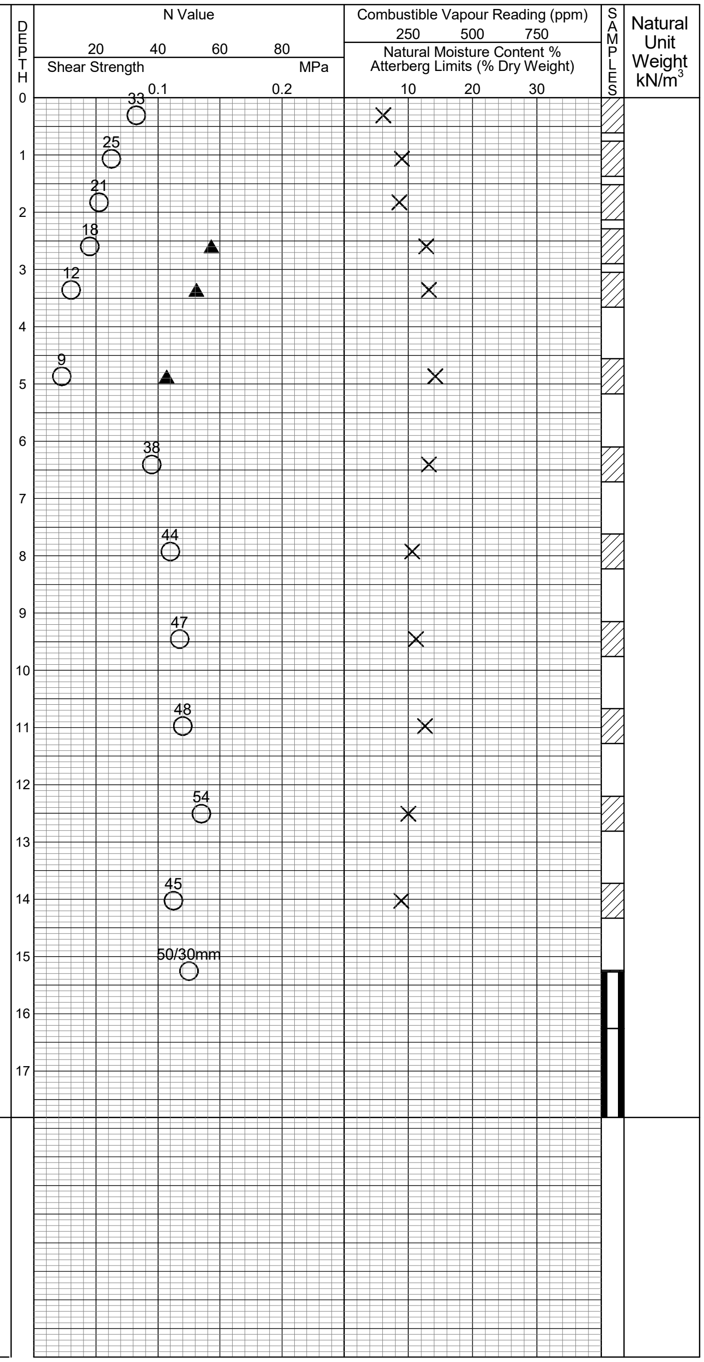
Auger Sample
SPT (N) Value
Dynamic Cone Test
Shelby Tube
Field Vane Test

Combustible Vapour Reading
Natural Moisture
Plastic and Liquid Limit
Undrained Triaxial at % Strain at Failure
Penetrometer

Soil Description	ELEV. m
	95.15
PAVEMENT STRUCTURE - 165 mm asphaltic concrete	95.0
- over 535 mm granular material	94.5
FILL - silty sand to sandy silt, trace to some gravel; brown, moist	93.1
CLAYEY SILT TILL - trace gravel; brown/grey, moist, stiff to very stiff - becoming grey - becoming very moist	88.8
SILT TILL - trace to some clay, trace to some sand, trace gravel, numerous sand seams; grey, moist, dense to very dense	80.2
WEATHERED SHALE - grey, moist **RUN#1: 15.24 to 16.26 m** TCR = 73% RQD = 39% **RUN#2: 16.26 to 17.81 m** TCR = 89% RQD = 52%	77.3

END OF BOREHOLE

NOTES:
1. Groundwater monitoring well installed to 9.15 m depth.



Time	Water Level (m)	Depth to Cave (m)
On completion	N/A	Well
January 11, 2022	7.68	Well
January 13, 2022	6.63	Well
January 26, 2022	4.75	Well

exp.