Preliminary Geotechnical/ Hydrogeological Report

Proposed Commercial/Industrial Development
Claremont North Business Park Development
5435, 5455 and 5475 Old Brock Road
Pickering, Ontario

Prepared for:
S. Larkin Developments Inc.

Prepared by:
Stantec Consulting Ltd.
300 – 675 Cochrane Drive
Markham, L3R 0B8

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was retained by S. Larkin Developments Inc., (Larkin) to conduct a preliminary geotechnical and hydrogeological investigation for a proposed business park development located at the municipal addresses of 5435, 5455 and 5475 Old Brock Road in Pickering, Ontario, hereinafter referred to as the "Site".

This report documents the results of the preliminary geotechnical investigation and hydrogeological assessment of the property.

The purpose of this combined preliminary geotechnical investigation and hydrogeological assessment was to:

- Confirm the subsurface conditions on the Site;
- Characterize hydrogeological conditions within the property boundaries of the Site; and,
- Recommend preliminary approaches for the foundations, floor slab and parking lot construction for the proposed business park development.

It is understood and acknowledged that additional geotechnical investigation and hydrogeological assessment services will likely be required prior to finalization of a detailed design and construction of the ultimate preferred development, including sourcing a suitable potable water supply for the proposed development.

Use of this report is subject to the Statement of General Conditions provided in **Appendix A**.

2.0 SITE DESCRIPTION

The Site is located at the municipal addresses of 5435, 5455 and 5475 Old Brock Road which is located west of Brock Road and east of Old Brock Road and immediately south of the Uxbridge Pickering Townline in the Village of Claremont. The location is illustrated on the Borehole Location Plan included in Drawing No.1 in **Appendix B**. The property is comprised of a parcel of predominantly undeveloped land with several structures and a residence on the property. The subject property occupies a total area of approximately 43,731 m^2 (10.81 acres), which was developed based on the Site Plan A1.3 prepared by Caricari Lee Architects (Caricari Lee).

The property is generally triangular. The width (east-west) is approximately 188 m on the south boundary and the length (north-south) is approximately 404 m on the west boundary and 452 m on the east boundary (adjacent to Brock Road).

The ground surface cover across the bulk of the property consists of grass, weeds, and shrubs with a residence, agricultural operation, and several structures on-site.

The general ground surface topography is characterized as sloping down from northwest to southeast, with an overall gradient change in the Site of approximately 14 m.

3.0 PROPOSED DEVELOPMENT

The proposed development is anticipated to include a Gas Station with a drive thru with additional retail uses. Three (3) industrial buildings in addition to the existing buildings on site are to be also included. All the buildings are anticipated to be one-storey slab-on-grade structures.

Parking areas and access roads for light passenger vehicles will be constructed at the periphery of the proposed buildings with vehicular access to both Brock Road and Old Brock Road.

3.1 REGIONAL GEOLOGY

3.1.1 Overburden

The study area is situated within a physiographic region identified as the South Slope, in close proximity to the boundary of the physiographic region known as the Peel Plain, as identified by Chapman and Putnam (1984). The South Slope generally consists of sandy silt to silty sand textured soils and is characterized as the southern slope of the Oak Ridges Moraine extending for approximately 200 kilometers from the Niagara Escarpment to the Trent River. The Peel Plain generally consists of glacial till soils and is characterized as a level to undulating tract of clayey soils covering approximately 800 square kilometers across central portions of the Regional Municipalities of York, Peel, and Halton.

The Quaternary Geology of Southern Ontario Map 2556 indicates that the overburden in the region consists predominantly of glaciolacustrine deposits primarily comprised of silt to silty clay.

Quaternary Geology Toronto and Surrounding Area, Southern Ontario Map 2204 identifies the overburden in the area as Halton Till, having a silty clay texture, occasionally overlain by local patches of lacustrine silty clay of the Peel Ponds and alluvium.

3.1.2 Bedrock

Bedrock Geology of Ontario, Southern Sheet, Map 2544 indicates that the region is located in an area underlain by bedrock of the Georgian Bay Formation. The Georgian Bay Formation consists of shale, limestone, dolostone and siltstone, generally being grey shale with limestone interbedding.

3.2 SCOPE OF WORK

The proposed scope of services for the geotechnical investigation included the following:

- Advance four (4) boreholes across the Site to a depth of 18 m below existing grade. In addition, record the groundwater conditions, where encountered, at the time of drilling in the open boreholes. Installation of groundwater monitoring wells in the boreholes for measurement of groundwater levels.
- Excavate three (3) test pits to a depth of 2.0 m.
- Prepare a geotechnical report that included the following components:
  - Site Plan showing the borehole locations;
  - Borehole records;
  - Factual results of the investigation and conditions encountered;
  - Results of the geotechnical laboratory testing program; and,
  - Geotechnical comments addressing the following:
  - Discussion of unusual or problematic conditions identified or encountered during the investigation and their implication with respect to the planned scope of development;
  - Site preparation requirements;
  - General groundwater control requirements (temporary for construction and permanent) as warranted in consideration of the site conditions encountered;
  - Anticipated foundation type, foundation depths/elevations and bearing resistances and reactions for ULS and SLS with estimates of anticipated settlements;
  - Comments regarding floor slab design and construction;
  - Determination of the applicable Seismic Site Classification based on the overburden conditions encountered to the termination depth of the boreholes (extrapolation below the termination depths will consider the geological conditions identified on area mapping and reports);
  - Suitability of existing soil materials for reuse as backfill;
  - Frost susceptibility assessment of existing soils; and,
  - Typical asphalt pavement structure for the access laneways and parking lot.

The proposed scope of services for the hydrogeological assessment included the following:

- Completed installation of four (4) shallow and two (2) deep groundwater monitoring wells in conjunction with the geotechnical drilling program;
- Completed hydraulic testing of each of the six (6) newly installed groundwater monitoring wells;
- Completed groundwater quality sampling at five (5) newly installed groundwater monitoring wells;
- Instrumented the groundwater monitoring wells and two (2) private wells with automated pressure transducers to record groundwater level changes over time; and,
- Provide recommendations for sourcing / testing a suitable public potable water supply for the proposed development.

The locations of the boreholes, monitoring wells, and private wells are shown on Drawing No. 1 in **Appendix B**.

4.0 METHOD OF INVESTIGATION

4.1 PREPARATORY SERVICES

Prior to commencing the drilling and excavating investigation components, Stantec, in coordination with Larkin, undertook the following tasks:

- Various public utility companies were consulted to identify where public utilities were present in the area of intended drilling and excavations; and,
- Some public utility locate companies do not provide locating services on private property. In this respect, Larkin retained the services of a private utility locating company to mark and clear the locations of the investigation holes of possible buried services.

4.2 FIELD PROGRAM

Prior to commencing the field investigation, the various public utility companies were consulted to identify where public utilities crossed the property boundaries. In addition, a private locator was contracted to clear the boreholes and test pits of any on-site services.

The fieldwork for the drilling investigation was carried out on June 6 to 10, 13 to 17, and 18, 2016. Water quality sampling was completed on June 15, 2016, and hydraulic testing was completed on June 14 and 15 and July 19 and 20, 2016. Water level monitoring was conducted in June, July, and August 2016 and June 2017. A total of four (4) boreholes (BH1, BH4, BH7 and BH10) with corresponding groundwater monitoring wells (MW1-D, MW4-S/D, MW7-S and MW10-S/D) were installed and a total of three (3) test pits (TP1, TP2 and TP3) were advanced and excavated for this investigation at the locations shown on Drawing No. 1 in **Appendix B**.

As the investigation is preliminary in nature for discussion purposes, the boreholes were selected to obtain overall coverage of the Site for general site characterization.

The borehole depths were extended to a depth of 30 m in three (3) of the four (4) boreholes as a component of the Hydrogeological Investigation.

The boreholes were advanced using a track mounted drill rig equipped with 210 mm (outside diameter) hollow-stem augers. Stantec personnel recorded the conditions encountered in the boreholes. Where overburden soils were present, samples were recovered at regular intervals using a 50 mm (outside diameter) split-tube sampler by conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in ASTM specification D1586. All soil samples recovered from the boreholes were placed in moisture-proof bags and returned to our laboratory for detailed geotechnical classification and testing as required.

A rubber-tire backhoe was used to excavate three (3) test pits to obtain subsurface information and to visually assess the quality of the sub-grade soil conditions. Specifically, the test pits would provide additional opportunity to view the conditions associated with:

1. The stability of open cut excavations;
2. Presence of earth fills;
3. Presence of seepage and/or static groundwater; and,
4. The presence of cobbles and/or boulders.

The test pits would also provide an opportunity to collect bulk samples for laboratory testing purposes. The test pits were intended to be excavated to depths in the order of 2.0 m.

The test pits were excavated at the locations shown on Drawing No. 1 in **Appendix B**.

The test pits were backfilled with the excavated soil upon completion. The backfill was tamped in place using the excavator bucket and the ground surface roughly leveled with the surrounding ground.

The conditions encountered in the boreholes and test pits were recorded by a geotechnical technician. Samples of the materials encountered in the boreholes and test pits were collected, placed in moisture-proof containers, and transported to Stantec's laboratory.

4.3 GROUNDWATER MONITORING

In order to characterize hydrogeologic conditions within overburden material encountered at the Site, Larkin retained Profile Drilling Inc. (Profile) to drill geotechnical boreholes and install five (5) monitoring wells at three (3) locations (MW1, MW7, and MW10) at the Site, with one (1) shallow well completed at MW1, and two (2) separate shallow (-S) and deep (-D) monitoring wells completed at each of MW7 and MW10 in accordance with Ontario Regulation (O. Reg.) 903. As a result of artesian conditions and heaving sands within the borehole for the originally planned deep monitoring well, MW7-D, the borehole was abandoned and decommissioned. The shallow and deep monitoring well pair, MW4-S/D, was drilled and installed as its replacement.

The water levels recorded in the monitoring wells are presented in Figure No. 4 in **Appendix B**. The locations of the monitoring wells were recorded by the drilling contractor in accordance with the Ontario Ministry of the Environment Regulation 903.

A D-50 drill rig equipped with 210 mm outside diameter (OD) hollow stem augers (HSA) and a tri-cone bit was used to advance the borehole, with split spoon soil samples obtained every 0.76 m. Monitoring wells were completed between June 7 and July 18, 2016 and are shown on Figure 2. All drilling and monitoring well installations were observed by Stantec personnel. Monitoring well installation details are summarized in Figure No. 6, and presented on the borehole logs in **Appendix B.**

The shallow (-S) monitoring wells were installed using 210 mm OD HSA. Installation depth was selected such that the monitor extended to near the proposed base of the open entry / exit pit excavations in order to assess potential dewatering requirements for the excavations, and ultimately was completed at depths of 4.9 m below ground surface (BGS) for MW1 and 4.6 m BGS for MW4-S, MW7-S, and MW10-S. The nature and stratigraphy of the geologic material encountered was considered when selecting the screened interval depth.

The deep (-D) monitoring wells at MW4-D and MW10-D were installed using 210 mm OD HSA. Installations for MW4-D and MW10-D were completed at depths of 29.0 m BGS and 26.5 m BGS, respectively. Artesian water level conditions were observed within MW4-D, with a measured water level of 1.46 m above ground surface (AGS) on August 30, 2016.

Monitoring wells MW1, MW4-S, and MW10-S/D were completed as 51 mm diameter PVC wells and were screened using 3.05 m length No. 10 slot Schedule 40 PVC screen. Monitoring well, MW4-D, was completed as a 38 mm diameter PVC well and was screened using a 3.05 m length, 64 mm diameter pre-packed sand screen. The pre-packed screen was used in order to effectively install the monitoring well within the heaving sand conditions encountered during drilling. The annular space of each hole was backfilled with No. 02 grade silica sand, from 0.15 m to 0.61 m below the bottom of the screen to a height of 0.61 m above the top of screen. The remainder of the annular space above and below the sand filter pack was filled with time-release coated bentonite chips (Pel-Plug) and bentonite chips (Holeplug). Bentonite grout was used above the bentonite chips and concrete was used within 0.46 m to 0.61 m of ground surface to seal the holes. Above ground casings were installed at all monitoring wells.

Upon installation of the groundwater monitoring wells, Stantec completed *in-situ* hydraulic conductivity testing at each of the six (6) newly installed wells, obtained representative groundwater quality samples from four (4) of the wells, and instrumented each well with a pressure transducer to record groundwater levels within each of the monitoring wells and within three (3) nearby private water supply wells. The results of the monitoring and sampling are provided in the following sections.

Upon completion of the investigation, ownership of the standpipe installations will reside with Larkin. It is noted that decommissioning of monitoring wells must be conducted in accordance with Regulation 903/90 as amended 128/03. It is presumed that decommissioning of the monitoring wells can be addressed via the scope of work established for the general contractor responsible for developing the site.

4.4 GROUNDWATER QUALITY SAMPLING

Stantec collected groundwater water quality samples from monitoring wells MW1, MW7-S, MW10-S, and MW10-D on June 15, 2016 for the purposes of assessing groundwater as a potential use as drinking water.

Stantec developed the monitoring wells after installation by purging a minimum of ten (10) well volumes from each well to allow formation water to flow through and prepare the wells for sampling. Groundwater samples were collected from the wells using dedicated water sampling equipment that consisted of 16 mm outside diameter, high-density plastic sampling tubing attached to a ball-check assembly. The tubing was lowered to approximately the centre of the screen for sample collection. Samples were collected from the dedicated tubing directly into appropriate laboratory supplied sample containers. The samples were not field-filtered and represent total metals concentrations.

The samples were submitted to Maxxam Analytics Inc. (Maxxam), an independent, certified laboratory for groundwater quality analyses using the Rapid Chemical Analysis program (RCAp) Drinking Water suite of parameters.

Water quality results are presented in Figure No. 7 in **Appendix B** and are compared to the Ontario Regulation 169/03 Drinking Water Quality Standards (ODWQS) criteria.

4.5 SURVEY

Borehole and test pit locations were surveyed in the field by Stantec using the base co-ordinate system. The ground surface elevation at the borehole is referenced to a Geodetic Datum.

Borehole and test pit elevations were taken from spot elevations on the Topographic Survey Plan prepared by Lloyd & Purcell Ltd., dated March 30, 2016.

Ground surface elevation at the borehole and test pit locations is provided on the Borehole and Test Pit Records attached in **Appendix C**.

4.6 SOIL LABORATORY TESTING

All soil samples returned to the laboratory were subjected to detailed visual examination and classification.

Grain size distribution, Atterberg Limits and moisture content tests were conducted on representative samples of the soils obtained from the investigation. The results of the laboratory tests are discussed in the text of this report and are provided on the Borehole Records in **Appendix C** and the figures included in **Appendix D**.

Unless requested in advance, all samples will be stored in our laboratory for a period of two months, following completion of the field work.

5.0 RESULTS OF INVESTIGATION

5.1 GENERAL INFORMATION

The subsurface conditions encountered in the boreholes are shown on the Borehole and Test Pit Records provided in **Appendix C**. An explanation of the symbols and terms used on the Borehole and Test Pit Records is also provided in the appendix.

In general, the ground surface cover consisted of topsoil. An alluvial deposit consisting of organic silty clay was encountered underlying the topsoil in one location. Fill materials consisting of silty clay, sandy silt, silty sand and sand and gravel were encountered underlying the surface cover in some of the boreholes and test pits. Native silty clay till, silty clay, sandy silt till, silty sand till, silty sand, sand, silt with sand and silt were encountered below the ground surface cover, alluvial deposit, and fill.

Bedrock was not encountered to the termination depth in any of the boreholes.

Groundwater was encountered in all the boreholes during drilling and was recorded at depths ranging from approximately from 4.18 m BGS to 1.46 m AGS.

The test pits remained dry upon completion of the excavations.

A general overview of the soil and groundwater conditions encountered in the boreholes is provided below.

5.2 GROUND SURFACE COVER

The ground surface cover at the borehole locations typically consisted of rough grass/weed cover or landscaped vegetation.

Topsoil

Ground surface cover at boreholes BH1, BH4, BH7 and BH10 and test pits TP1, TP2 and TP3 consisted of rough grass/weed cover with supporting topsoil. The thickness of topsoil was approximately 100 mm to 700 mm. A layer of topsoil, approximately 600 mm and 900 mm thick was encountered underlying the earth fill in boreholes BH7 and BH10 and test pit TP3 and extends to a depth of approximately 2.4 m to 3.2 m below the existing grade.

5.3 ALLUVIAL SILTY CLAY DEPOSIT

An alluvial deposit consisting of a dark grey organic silty clay deposit, 1.4 m thick, was found underlying the topsoil in borehole BH7 and extends to a depth of 3.4 m from the ground surface. The alluvial organic silty clay's dark grey colour indicates that it contains an appreciable amount of roots and humus, and was also laminated with peat seams containing decayed vegetation and wood debris.

The alluvial organic silty clay was assessed as wet based on visual and textural examination of the samples in the field.

5.4 EARTH FILL

Earth fill material consisting of layers of silty clay, sandy silt, silty sand, and sand and gravel was encountered underlying the ground surface cover in all the boreholes and test pits TP2 and TP3. The samples obtained from the boreholes contained occasional to frequent topsoil and organic inclusions. The fill material extended to depths of approximately 1.5 m to 5.2 m below existing grade.

The N-values ranged from 2 to 37, with a median of 6 blows were obtained from the SPTs advanced in the fill materials. The cohesive fill material (silty clay) had a consistency typically ranging from very soft to hard and the cohesionless materials (sandy silt, silty sand and sand and gravel) were loose. The values show the fill materials were loosely placed with a nominal to some degree of compaction.

Based on visual and textural examination, the fill materials were assessed as moist to wet. The results of the moisture content tests ranged from 9% to 30%, with an average of 15%.

Grain size analyses tests were completed on three (3) samples of the earth fill consisting of silty clay, sandy silt, and silty sand soils. The results of the tests are shown in Table 5.1 below.

Table 5.1: Gradation Analysis Test Results for Earth Fill Soils

Borehole & Test Pit No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS2	1.1	8	46	33	13
BH10	SS5	4.1	6	37	43	14
TP2	GS2	1.1	0	18	64	18

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.5 SILTY CLAY TILL

A soil stratum of glacial silty clay till was encountered underlying the topsoil in test pit TP1 and in the lower zone of the stratigraphy underlying the silt in boreholes BH7 and BH10. The glacial till consists of silty clay and contains seams and interstratified layers of sand and gravel with occasional cobbles and boulders increasing with depth. The silty clay till extended onto a silty sand till deposit.

The silty clay till was brown in colour, changing to grey with depth. Occasional fine sand lenses and partings were observed in some of the glacial till samples, some of which were wet with seepage. The consistency of the silty clay till soil was typically assessed as stiff to hard, being generally very stiff.

Based on visual and textural examination, the silty clay till was assessed as moist. The results of the moisture content tests ranged from 3% to 12%, with an average of 10%.

Grain size analyses tests were completed on four (4) samples of the silty clay till soils. The results of the tests are shown in Table 5.2 below.

Table 5.2: Gradation Analysis Test Results for Silty Clay Till Soils

Borehole & Test Pit No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS11	8.1	0	11	66	23
BH10	SS13	11.0	1	12	68	19
TP1	GS3	0.8	3	37	41	19
TP1	GS5	1.1	8	31	43	18

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.6 SILTY CLAY

The predominant stratum of silty clay was encountered in all the boreholes throughout the upper and lower zones of the stratigraphy underlying the earth fill and alluvial deposit and interstratified within the sands and silts.

The silty clay contains wet sand seams and displays a faintly visible varved structure.

N-values of 2 to 44, with a median of 15 blows were obtained from the SPTs advanced in the silty clay. The cohesive silty clay encountered in the boreholes had a consistency typically ranging from very soft to hard, generally being firm to stiff.

Based on visual and textural examination, the silty clay was assessed as moist. Laboratory test results on representative samples of the silty clay yielded moisture contents of approximately 13% to 16%, with a median of 15%. The native silty clay soil is considered highly frost susceptible and has a high potential for soil adfreeze.

Grain size analyses tests were completed on two (2) samples of the silty clay soil. The results of the tests are shown in Table 5.3 below.

Table 5.3: Results of Grain Size Analysis Tests on Samples of the Silty Clay

Borehole No.	Sample	Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS18	18.6	0	5	67	28
BH10	SS20	21.6	0	21	60	19

The results of the Grain Size Analysis tests on samples of the native silty clay are illustrated on Figure 1 in **Appendix D**.

Atterberg Limits Tests were completed on a portion of the samples noted above. The results of the test were as follows:

Table 5.4: Results of Atterberg Limits Tests on Samples of the Silty Clay

Borehole No.	Sample	Median Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Moisture Content (%)
BH10	SS18	18.6	19	14	5	19
BH10	SS20	21.6	16	12	4	16

The results of the Atterberg Limits Tests conducted on samples of the native silty clay are illustrated on Figure 2 in **Appendix D**.

Based on the results of the laboratory testing as discussed herein, this soil can be classified as silty clay with low plasticity. Based on the Unified Soil Classification System the samples tested are classified as Lean Clay (CL-ML). Additional variations of this soil are likely present in the stratigraphy across the site. For purposes of convenience in this report, this soil is described as silty clay.

5.7 SANDY SILT TILL

A localized sandy silt till and was encountered in borehole BH10 underlying the sandy silt fill. The glacial till consists of sandy silt and contains seams and interstratified layers of sand and gravel with cobbles and boulders increasing with depth. The sandy silt till is grey in colour.

Occasional fine sand lenses and partings were observed in some of the glacial till samples, some of which were wet with seepage.

The compactness of the sandy silt till soil based on the N-Values obtained from the SPTs, was assessed as compact.

Based on visual and textural examination, the sandy silt till was assessed as moist. Laboratory test results indicated that the moisture content was approximately 8% and 10%.

Grain size analyses tests were completed on two (2) samples of the sandy silt till soil. The results of the tests are shown in Table 5.5 below.

Table 5.5: Gradation Analysis Test Results for Sandy Silt Till Soils

Borehole No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS8	5.6	3	34	45	18
BH10	SS10	7.2	1	26	58	15

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.8 SILTY SAND TILL

A localized stratum of silty sand till was encountered in the lower zone of the stratigraphy underlying the silty clay and silty clay till in boreholes BH4, BH7 and BH10. The silty sand till typically contained some clay, and a trace to some gravel.

Based on the N-values obtained from the SPT's, the state of compactness of the silty sand till was assessed as very loose to compact. Based on visual and textural examination, the silty sand till was assessed as moist. The results of the natural moisture content tests were approximately 9% and 12%.

A grain size analysis test was completed on one (1) sample of the silty sand till soil. The results of the test are shown in Table 5.6 below.

Table 5.6: Gradation Analysis Test Results for Silty Sand Till Soils

Borehole No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS14	12.3	2	71	21	6

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.9 SILTY SAND

A localized stratum of silty sand was encountered in the lower zone of the stratigraphy underlying and interstratified within the silty clay in boreholes BH7 and BH10. The silty sand typically contained a trace of clay with occasional silt and silty clay layers.

Based on the N-values obtained from the SPT's, the state of compactness of the sand was assessed as very loose to very dense. The very loose sand deposit occurred in borehole BH7 at depths between 4.7 m to 5.5 m, and between 12.2 m and 13.3 m below ground surface.

Based on visual and textural examination, the sand was assessed as moist to wet.

A grain size analysis test was completed on one (1) sample of the silty sand soil. The results of the tests are shown in Table 5.7 below.

Table 5.7: Gradation Analysis Test Results for Silty Sand Soils

Borehole No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS19	24.7	0	74	22	4

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.10 FINE SAND

A localized stratum of fine sand was encountered in the lower zone of the stratigraphy underlying the silty clay and silty sand in boreholes BH4 and BH7 and was found extending beyond the investigated depths of 29.6 m and 31.9 m in both boreholes. The fine sand typically contained a trace of clay with occasional silty sand layers.

Based on the N-values obtained from the SPT's, the state of compactness of the fine sand was assessed as very loose to very dense. The very loose fine sand deposit occurred in borehole BH4 at depths between 24.3 m to 25.0 m below ground surface.

Based on visual and textural examination, the fine sand was assessed as moist to wet.

5.11 SILT WITH SAND

A localized stratum of silt with sand was encountered in the lower zone of the stratigraphy underlying the silty clay in borehole BH10 and was found extending beyond the investigated depth of 30.3 m. The silt with sand typically contained some clay with occasional silt and silty clay layers.

Based on the N-values obtained from the SPT's, the state of compactness of the silt with sand was assessed as loose to compact.

Based on visual and textural examination, the silt with sand was assessed as moist to wet. The results of the natural moisture content tests ranged from approximately 10% to 22%, with an average of 15%.

A grain size analysis test was completed on one (1) sample of the silt with sand soil. The results of the tests are shown in Table 5.8 below.

Table 5.8: Gradation Analysis Test Results for Silt with Sand Soils

Borehole No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS24	27.9	0	26	56	18

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.12 SILT

A localized stratum of silt was encountered in the lower zone of the stratigraphy underlying the silty sand and silty sand till in boreholes BH7 and BH10. The silt typically contained a trace of gravel with some clay and sand. The silt contained occasional silty clay layers.

Based on the N-values obtained from the SPT's, the state of compactness of the silt was assessed as compact to very dense.

Based on visual and textural examination, the silt was assessed as moist. The result of the natural moisture content test on one sample was 9%.

A grain size analysis test was completed on one (1) sample of the silt. The results of the tests are shown in Table 5.9 below.

Table 5.9: Gradation Analysis Test Results for Silt Soils

Borehole No.	Sample No.	Sample Median Depth (m)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH10	SS15	14.0	0	11	73	16

The results of the tests are also shown on Figure 1 in **Appendix D**.

5.13 HYDRAULIC TESTING

Stantec completed *in-situ* hydraulic conductivity testing at all six (6) newly installed monitoring wells on June 14 and15 and July 19 and 20, 2016. Stantec temporarily instrumented each well with a pressure transducer and completed one rising head test at each well. The results were analyzed using the Bouwer and Rice (1976) solutions for unconfined aquifers (MW1, MW4-S, MW7-S, and MW10-S) and confined aquifers (MW4-D and MW10-D), as provided in the software package AQTESOLV™. Analytical results indicate that the horizontal hydraulic conductivity for the sand to silty clay material present within the well screened intervals ranges from 1.0×10^{-8} m/s to 5.9×10^{-6} m/s, with a geomean of 4.3×10^{-7} m/s.

Solutions for the hydraulic conductivity testing results are summarized in Figure No. 6 in **Appendix B**. The values determined during the field testing were generally consistent with published literature values for the various screened materials (Fetter, 1994).

5.14 GROUNDWATER QUALITY RESULTS

Stantec collected representative groundwater water quality samples from 3 shallow and 2 deep monitoring wells MW1, MW7-S/D and MW10-S/D on June 15, 2016 for the purposes of determining water quality for potential potable water supply for the proposed new development.

Water quality results are presented in Figure No. 7 in **Appendix B**, summarized below, and are compared to Ontario Drinking Water Quality Standards (ODWQS) to assess the use of groundwater as a potential potable water supply. Water quality was found to exceed the ODWQS for the following criteria:

- General Chemistry: met all ODWQS criteria with the exception of the following:
  - Total Alkalinity (MW7-S/D, and MW10-S)
  - Dissolved Chloride (MW10-S/D)
  - Dissolved Organic Carbon (All Wells)
  - Hardness (All Wells); and
  - Total Dissolved Solids (MW7-S/D, and MW10-S/D);

- Total Metals: met all ODWQS criteria with the exception of the following:
  - Aluminum (All Wells);
  - Iron (All Wells);
  - Manganese (All Wells); and,
  - Sodium (All Wells).

Water quality results from both shallow and deep monitoring wells indicate raw groundwater quality does not meet the ODWQS criteria; however, many of the reported exceedances are typically encountered in sources within native soils in Southern Ontario and may be improved significantly with treatment. The presence of elevated chloride and sodium in MW10S/D may be related to road salt use. Follow-up water quality analyses are recommended following further well development during pumping tests, with analyses for total suspended sediment, total metals and dissolved metals analyses included.

5.15 GROUNDWATER LEVEL MONITORING RESULTS

As part of the groundwater level monitoring program at the Site, each monitoring well was instrumented with a pressure transducer to record water level readings at hourly intervals following the completion of hydraulic conductivity testing. Three (3) existing private wells (2 on site, and 1 offsite to the south) were also instrumented with pressure transducers by a licensed well contractor from D&S Water Well Service Ltd., (D&S) under the supervision of Stantec in June 2016. Stantec measured groundwater levels manually during Site visits using a Solinst 30 m Water Level Meter between June 2016 and June 2017 from within the newly installed monitoring wells. Data was downloaded from the pressure transducers within the monitoring wells on August 30, 2016 and June 22, 2017. D&S measured groundwater levels manually from the private wells on September 19, 2016 and June 22, 2017, and Stantec downloaded the data from the pressure transducers installed within the wells at the same time.

Shallow and deep groundwater elevation contours are presented in **Appendix B** on Figure No. 2 and Figure No. 3, respectively. Groundwater level hydrographs are presented on Figure No. 4, with private well hydrographs presented on Figure No. 5. Temperature and climate data from nearby Environment Canada Oshawa Climate Station, supplemented with data from Environment Canada Uxbridge Climate Station, are presented along with water level data on each of the hydrograph figures.

The shallow groundwater contours presented on Figure No. 2 indicate that shallow groundwater flows to the south-east, generally following topography. Deep groundwater contours, presented on Figure No. 3, indicate that deeper groundwater flow direction beneath the site is to the south-west. The groundwater flow direction was consistent throughout the monitored period.

The continuous water level data allowed the interpretation of seasonal groundwater level variations and responses to precipitation events. Twelve months of continuous water level data are available from June 2016 to June 2017. Hydrographs of the continuous water level data for on-Site monitoring wells are presented in Figure No. 4. Over the monitored period, the groundwater levels were observed to remain relatively stable throughout late summer and fall followed by an increase in mid-December in relation to increased precipitation associated with the fall and/or decreased private well pumping. The seasonal water level variation was muted at MW7 compared to MW1, MW4-S, and MW10-S where a 1 m (MW1 and MW10-S) to 4 m (MW4-S) seasonal water level response was observed. A water level response to precipitation events of about 0.75 m and 1.0 m was observed at MW1 and MW4-S, respectively. The water level response to precipitation events at MW7 and MW10-S was muted and generally less than 0.25 m.

The groundwater level response observed at MW10-D shows some daily fluctuations not observed in the other monitoring wells. The fluctuating response may be due to well use in nearby deep private wells. As well, MW1-S shows a correlation in increasing groundwater level associated with decreased pumping at the north well in early January 2016. Selecting a location for a pumping test and potential Site water supply will need to consider potential well interference with nearby private wells; with the northern and eastern sides of the Site preferred based on currently available, pending final Site development plans.

The ground surface at each monitored location is also presented on Figure No. 4 for comparison to the depth of groundwater. The groundwater level was generally at ground surface in the southern portion of the site (MW7 and MW10-S) and within 1 m of ground surface within the northern portion of the site (MW1 and MW4-S) throughout the winter and spring. In the summer and fall, the groundwater level was generally 0.5 m (MW7) to 4.25 m (MW1) below ground surface.

Groundwater levels in the deeper overburden responded differently than shallow overburden suggesting the two units are not hydraulically connected. The deep groundwater monitoring wells were completed in coarser grained material relative to the overlying confining finer grained material composed of predominantly silt and clay. The deep groundwater levels presented in Figure No. 4 were generally consistent with less than a 1 m groundwater level variation throughout the monitored period.

On-Site monitoring well hydrographs presented on Figure No. 4 indicate an upward vertical hydraulic gradient at MW4-S/D ranging from 0.14 m/m in fall 2016 to 0.11 m/m in winter 2017; while at MW10-S/D, the vertical hydraulic gradient was downward at 0.03 m/m in fall 2016 and 0.08 m/m in winter 2017. The upward hydraulic pressure observed at MW4-D, with recorded water levels in the deep well above ground surface (flowing artesian) is the likely source of groundwater to the southeast portion of the Site, where soil conditions at surface were observed to be saturated during site visits in the vicinity of the culvert underneath Brock Road.

Private well hydrographs show typical daily fluctuations due to well use and slight responses due to dry summer conditions recorded during the summer months. Despite precipitation over the summer months being recorded as reduced by approximately 50% as compared to the same period in 2015, the water level recorded in the North Private Well as well as the South Off-Site Well remained relatively consistent through the monitoring period.

The deeper aquifer within which the deep monitoring wells and nearby private wells are installed are likely the preferred target of potential water supply for the development site. A pumping test complete with monitoring of on-Site monitoring wells and nearby private wells will be required to characterize hydraulic conditions of this potential source of water as well as determine potential private well interference and the mitigation options that may be employed to protect groundwater resources.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 SUMMARY AND EVALUATION OF EXISTING CONDITIONS

In general, the subsurface stratigraphy across the Site can be described as follows:

- Ground surface cover consisting of rough grass/weed cover vegetation; underlain by,
- Alluvial deposit consisting of a dark grey organic silty clay; underlain by,
- Layers of silty clay, sandy silt, silty sand, and sand and gravel fill; underlain by,
- Native stiff to hard, generally very stiff silty clay till and very soft to hard, generally firm to stiff silty clay; underlain by,
- Native compact sandy silt till and loose to compact silty sand till stratum; and underlain and interstratified by very loose to very dense silty sand and sand, loose to compact silt with sand and compact to very dense silt.

There was significant deposit of fill material reveled in the investigation holes advanced at locations within the proposed business park development. The fill material was approximately 1.5 m in thick in the north portion of the property increasing in depth to 5.2 m in thickness in the southwest corner of the property.

Bedrock was not encountered in the current investigation.

Groundwater was encountered in all the boreholes during drilling. The groundwater was recorded at depths ranging from approximately from 4.18 m BGS to 1.46 m AGS.

The test pits remained dry upon completion of the excavations.

Shallow groundwater contours indicate horizontal groundwater flow is to the south-east, generally following topography. Deep groundwater contours indicate that deeper horizontal groundwater flow direction beneath the site is to the south-west. Static groundwater levels in the monitoring wells from August 30, 2016 ranged from 4.18 m BGS to 1.46 m AGS, with artesian conditions recorded in the deeper well on measured from the northeast side (MW4-D) of the Site.

Groundwater levels recorded in on-Site monitoring wells indicate an upward hydraulic vertical gradient at MW4-S/D of 0.14 m/m, while at MW10-S/D, the vertical gradient is downward at 0.05 m/m. The upward hydraulic pressure observed at MW4-D, with recorded water levels in the deep well above ground surface (flowing artesian) is the likely source of groundwater to the southeast portion of the Site, where soil conditions at surface were observed to be saturated during site visits.

Water quality results from both shallow and deep monitoring wells indicate raw groundwater quality does not meet the ODWQS criteria; however, many of the reported exceedances are typically encountered in sources within native soils in Southern Ontario and may be improved significantly with treatment.

6.2 GEOTECHNICAL CONSIDERATIONS AND CONSTRAINTS

The following general geotechnical considerations and constraints are set forth with respect to the conditions encountered in the current investigation and the proposed scope of development as described herein. Additional comments and recommendations pertaining to all the items noted are included in the subsequent sections of this report.

- It is anticipated that portions of the associated infrastructure will be decommissioned as a component of the re-development of the Site. Excavations created through the demolition and decommissioning process should be backfilled with approved, compacted engineered fill materials;
- Groundwater was measured in the boreholes throughout the site in the sand and silt deposits at depths of approximately from 4.18 m BGS to 1.46 m AGS with recorded water levels in the deep well above ground surface (flowing artesian);
- It is anticipated that some minor to major sub-excavation and replacement of existing poor quality fill material will be required depending on the finalized locations of the building envelopes. Minor cuts and some minor fine grading, to develop general elevations consistent with developing the proposed finish floor elevations will be also required;
- The existing fill material is not suitable for the support of shallow strip and spread footings. To upgrade the fill to support foundations it is suggested the fill materials could be removed and replaced with engineered fill for normal footing construction with the foundation designed with a Serviceability Limit States (SLS) reaction of 150 kPa and an Ultimate Limit States (ULS) resistance of 225 kPa;
- It is required to remove the existing topsoil and alluvial deposit as a component of site preparation activities. The surface cover topsoil thickness varied from approximately 100 mm to 700 mm. A layer of buried topsoil, approximately 600 mm and 900 mm thick was encountered underlying the earth fill in boreholes 7 and 10 and extends to a depth of approximately 2.4 m to 3.2 m below the existing grade. The alluvial deposit consisting of a dark grey organic silty clay deposit, 1.4 m thick, was found underlying the topsoil in one location and it extends to a depth of 3.4 m from the ground surface;
- Should the buried topsoil and alluvial deposit remain in-place following the site preparation and building construction, measures should be implemented as the organic materials will emit methane gas as they decay which can potentially result in health and safety concerns if not properly ventilated. An under slab ventilation system should be incorporated into the design of the buildings;

- The use of conventional spread and strip footing foundations founded on the native silty clay is a practical foundation option for the planned scope of development within the locations of boreholes 1 and 4;
- Based on the findings in the areas of boreholes 7 and 10, earth fill approximately 3.4 m to 5.2 m thick was encountered. As an alternative to the sub-excavation of the unsuitable earth fill and replacement with engineered fill or deep foundations consisting of caisson or steel driven pile foundations, ground improvement techniques could be utilized to support the foundations of the buildings using rammed aggregate piers or dynamic compaction;
- Loose and very soft soils with artesian and flowing conditions were encountered below depths of 10 m below existing grade. Caution should be exercised during potential future caisson construction;
- The native stiff to hard clay to silty clay will provide a suitable base for the installation of small-size tanks. The uplift pressure due the presence of shallow groundwater must be considered in the design of the tanks assuming the groundwater at the ground surface;
- There are areas that will require some sub-excavation of existing fill materials and replacement with approved fill consistent with the recommendations provided above for conventional slab-on-grade floor. Should the existing fill remain in place for building construction, a structural slab should be considered in areas where deep fill was encountered;
- Generally, the existing fill is suitable for conventional pavement construction; however, there are some areas that will require some sub-excavation and replacement with approved fill for parking lot construction. It is suggested the prepared parking lot subgrade should be surface compacted, proof-rolled and inspected by a qualified Geotechnical Engineer. Any soft or deleterious areas or topsoil identified should be rectified accordingly; and,
- The use of conventional asphalt pavement for parking areas of heavy duty vehicles is considered suitable for the Site.

6.3 SITE PREPARATION

6.3.1 Erosion and Sediment Control

A site-specific and project-specific erosion and sediment control program should be implemented prior to commencement of earthwork activities on the Site and extending throughout the period of construction.

Due to the predominantly fine grained nature of the fill materials and native soils, the surface of the fill materials and native soils will become muddy and soft, and loss of strength will occur, when exposed to precipitation and surface runoff. This condition will be exacerbated under the application of loads from traffic and construction vehicles and operations.

It is recommended that the erosion and sediment control program contain, in part, a temporary grading plan which will promote ground surface runoff away from areas of planned development and construction activity, to be collected in low-lying areas or temporary ditches and swales.

6.3.2 Demolition and Decommissioning

All existing infrastructure, including foundations and buried service, associated with the existing 5475 Old Brock Road residence will require decommissioning and demolition.

Given the age of the Site and the apparent array of buried services across the Site, it must be anticipated that decommissioning, demolition, and excavation (as discussed in detail in a subsequent section of this report) may encounter unanticipated, unknown and unrecorded buried infrastructure. All infrastructure encountered must be removed to below a minimum of 1 m below the underside of the foundations for the new buildings.

The requirement to backfill any/all trenches and excavations created through the decommissioning and demolition activities is addressed in detail in the Grading Earthworks & Ground Improvement section below.

6.3.3 Cutting, Clearing & Stripping

Existing trees, bushes, shrubs and similar will require removal from all areas of planned development.

The existing grass, underlying topsoil and alluvial deposits must be stripped from all areas of planned development. Based on the conditions observed in the boreholes and test pits, the thickness of organic ground surface cover and topsoil to be stripped is anticipated to range from approximately 100 mm to 700 mm and a layer of buried topsoil, approximately 600 mm and 900 mm thick was encountered underlying the earth fill in in boreholes BH7 and BH10 and test pit TP3 and extends to a depth of approximately 2.4 m to 3.2 m below the existing grade. The alluvial deposit, 1.4 m thick, was found underlying the topsoil in borehole 7 and it extends to a depth of 3.4 m from the ground surface.

6.3.4 Grading and Earthworks

6.3.4.1 General Overview

The earthworks program (grading, cuts, and fills) should be designed in advance to consider and address: the time of year of execution; prevailing weather conditions; storm-water management control; types of soils and fill materials intended for use/reuse; and, excavation, handling, placement, and compaction requirements.

All buried infrastructure encountered (including pipes, utilities, vaults, slabs, and similar) must be removed to below the level of the underground storage tanks and/or to a minimum of 1 m below the underside of the foundations for the new buildings.

6.3.4.2 Engineered Fill

As indicated in Section 6.1, there was significant deposit of fill material revealed in the investigation holes advanced at locations within the proposed business park development. An option of engineered fill throughout the Site can be considered as an overall foundation alternative.

The compactness/consistency of the existing fill material is variable and is not suitable for the support of shallow strip and spread footings. The fill material within the building envelopes plus 3.0 m beyond the periphery should be removed (sub-excavation) and disposed of at a Landfill and/or reused where suitable and replaced with compacted engineered fill. The depth of sub-excavation anticipated to be required is in the order of 1.5 m in the north portion of the property increasing in depth to 5.2 m in thickness in the southwest corner of the property below existing grade.

Free groundwater was encountered at approximately 4.25 m BGS to 0.5 m BGS in the monitoring wells. It is therefore anticipated that excavations for engineered fill will extend to below the groundwater level in which case a dewatering program will be required for the operations.

The exposed sub-grade surface should be inspected during stripping and sub-excavation activities for the presence of deleterious materials, organics, or loose/soft or wet zones. If deleterious or organics are observed, or loose/soft or wet conditions are present, these areas should be sub-excavated and the excavated material replaced with engineered fill in accordance with the recommendations provided below.

Following completion of the required stripping and sub-excavation as noted above, the exposed surface of the native soils must be proof rolled and compacted using large, vibratory compaction equipment with a minimum static weight of ten tonnes. This will provide a uniform, compact surface that will minimize the potential for infiltration of precipitation and ground surface runoff and promote drainage at the ground surface. The proof rolling program should consist of a minimum of five passes per unit area to provide a uniform surface for construction.

All imported fill materials required to backfill the required sub-excavated areas, and to develop the design grades and elevations on the property should consist of OPSS Granular B – Type 1, OPSS Select Subgrade Material (SSM), or approved equal. Fill materials imported to the site must meet all applicable municipal, provincial, and federal guidelines and requirements associated with environmental characterization of the materials.

All materials placed as engineered fill should be placed in 200 mm thick loose lifts. Each lift should be uniformly compacted to achieve a minimum of 98% of the material's Standard Proctor Maximum Dry Density (SPMDD).

Upon completion of the removal of materials, footings founded on compacted engineered fill may be designed using values provided in Table 6.1.

Table 6.1: Footing on Compacted Engineered Fill

Footing Size	Factored ULS Resistance kPa	SLS Reaction kPa
Strip Footings		
0.6 m wide	225	150
0.45 m wide	225	150

The ULS value includes a resistance factor of 0.5, the SLS reaction has been calculated for 25 mm of settlement and presumes a minimum of 2 m of engineered fill below the footings.

With respect to the serviceability limits states application, the differential settlements should be less 20 mm (¾ of the total settlement specified above), consistent with industry standard analysis. However, it should be noted that considering the extent and depth of fill, especially in the middle to southern part of the Site, once the site grades and proposed elevations for the building structures and invert elevations are known, additional comments and recommendations can be provided to ensure that the total and differential settlements of the proposed building structures and services are limited to tolerable limits. It may necessary to delay the construction of settlement sensitive structures (i.e. – buildings, pavement surfaces, underground services, etc.) on the middle to the southern part of the property to ensure that any excessive settlements have taken place. An alternate to delaying construction is to preload the engineered fill area with surplus material to accelerate the rate of induced settlement. The settlement monitoring of the preloading can be carried out by survey instrumentation of settle monitoring plates installed in the engineered fill mantle.

Engineered fill in excess of 3 m is expected to consolidate and, due to the presence of soft and loose subsoil at the Site, a raise in grade is anticipated to cause consolidation. Structures should not be constructed for the first six months after completion of the engineered fill. Surcharging or consolidation monitoring may be required.

Typical footing dimensions for these applications include a strip footing width of 0.45 m to 0.6 m and a column footing dimension of 0.9 m and 1.2 m.

The Ontario Building Code and the guidelines in the Canadian Foundation Engineering Manual require any exterior footings and footings in unheated areas exposed to freezing temperatures be provided with a conservative minimum of 1.2 m of soil, or equivalent insulation, for adequate frost protection.

Where construction is undertaken during winter conditions, the footing subgrade should be protected from freezing.

The program for grading and earthworks should be designed in advance, and carefully executed in consideration of the time of year of execution, prevailing weather conditions, storm-water management control, and associated issues and concerns, and the intended end-use of the subject property as described herein.

6.4 REUSE OF SITE MATERIALS

6.4.1 Ground Surface Cover

Organic ground surface cover, topsoil and alluvial deposit are considered unsuitable for reuse in any form except for the topsoil which may be suitable for general landscaping. The scope of work described herein does not include testing of the topsoil to assess the potential for vegetative growth in this respect. If reuse for general landscaping is not intended, these materials should be removed from the Site to an approved off-site location.

6.4.2 Fill Materials

Portions of the existing fill materials, where available from excavations or cuts-to-grade, can be considered suitable for reuse in limited applications.

Any topsoil, topsoil inclusions, alluvial deposits and any debris or deleterious material should be removed from the fill materials to facilitate reuse. A portion of the fill materials was generally in a moist to wet condition; these fill materials will require drying to facilitate reuse.

The results of the natural moisture content tests on samples of the fill materials were in the range 9% to 30%. The lower end of the range is more representative of portions of the upper zone of the fill and the higher end of the range would be considered more representative of portions of the lower zone of the fill materials. In this respect, drying of the wetter fill materials will be required to facilitate handling, placement, and compaction, if reuse is contemplated.

For reference, a moisture content that exceeds the optimum moisture content by greater than approximately 2% will result in loss of strength and make these soils extremely difficult to handle, place, and compact. A moisture content that is too dry of optimum will make it difficult to achieve the required compaction.

Stockpiling of this soil, even on a temporary basis, should be avoided, as continued exposure to the natural environment, repeated cycles of wetting/drying, possible freeze-thaw cycles, and similar, will result in loss of strength and make this material practically impossible to handle, place, and compact.

The fill materials should be considered moderately to highly frost susceptible. These materials should therefore not be reused in any application where volume change as a result of exposure to freezing conditions in the presence of excess moisture would be detrimental to the serviceability of any proposed infrastructure or structures.

6.4.3 Native Soils

Generally, the predominate soils, silty clay till, sandy silt till and silty clay encountered during the investigation can be considered suitable for reuse as general engineered fill to develop design grades and elevations, or for use in backfilling service trenches. The results of the moisture content tests and visual inspection of the samples obtained from the investigation indicate that the till and silt soils should be suitable for reuse at the existing moisture content. It is noted, however, that coarser zones may exist in the till and these zones may contain perched groundwater. The quantity of groundwater and associated seepage may be considerable when initially encountered, but these zones are typically limited in extent and over a short duration, the seepage typically reduces considerably or stops entirely. Based on the results of the grain size test completed on representative samples of these soils, it is suggested that the till, silty clay, silt, silt with sand and silty sand, soils be considered to have a moderate to high frost susceptibility and should therefore not be used as perimeter foundations backfill, as granular base and sub-base materials, or for similar applications where development of frost could jeopardize the serviceability of the planned infrastructure.

6.5 EARTHWORKS IN ADVERSE WEATHER CONDITIONS

Additional precautions, effort, and measures may be required, when and where construction is undertaken during late fall, winter, and early spring construction when the temperature and climatic conditions have an adverse influence on the standard construction practices or during periods of inclement weather.

Given the overburden soils present on the site, issues associated with the time of year of construction may not be as prevalent as on other sites. However, the native soils may pose a concern, subject to the prevailing weather conditions. Provided that the perimeter of the site has a storm drainage swale installed and that the surface of the native soils (or engineered fill where the till is reused in this application) is sloped, compacted, and protected from exposure to excess moisture, freeze-thaw cycles, and loading from construction traffic, there should be limited concerns developed in this respect. With respect to all earthworks activities undertaken during the late fall through to late spring, when less-than-ideal construction conditions may prevail, the following comments are provided:

1. All of the engineered fill should consist of granular materials or select earth material approved by the geotechnical engineer. The intended area of fill should be clearly identified in the field prior to commencing the work.
2. Ramps or roads for access (see above for further consideration) must be constructed outside of the limits of intended fill.
3. Fill placement should be inspected by qualified field personnel on a full time basis under the supervision of a geotechnical engineer, with the authority to stop the placement of fill at any time when conditions are considered to be unfavourable.
4. Imported materials that contain ice, snow, or any frozen material should not be accepted for use.
5. Overnight frost penetration may occur, even in granular fill materials, where precipitation and ground surface runoff pools and accumulates, and freezing temperatures exist. Any frozen materials must be removed prior to placing subsequent lifts of engineered fill. Breaking the frost in-situ is not considered acceptable.
6. It may be necessary to stop the placement of engineered fill during periods of cold, where ambient temperatures are -5°C or less, exist.

The placement of engineered fill materials during cold weather conditions requires extra effort beyond that typical when better climatic conditions prevail. At any time where conditions are deemed unfavourable, the engineered fill operation must be suspended.

Additional considerations for heating of concrete, heating of forms and reinforcing steel, protection of concrete from freezing, and similar measures may also be required subject to climatic conditions at the time of construction.

Appropriate scheduling of the work may also require specific consideration and revision from the typical adopted. The scope of work intended may have to be reduced or adjusted, and/or only select construction activities be undertaken during specific climatic conditions. The areas of planned engineered fill may have to be reduced on a daily basis, the extent of excavations may have to be limited, with all excavating and associated backfilling completed without delay.

6.6 FOUNDATIONS

As the design is still preliminary in nature and still evolving, the following section provides various foundation options for the proposed development due to the presence of deep and variable earth fill and soft and loose materials encountered in some of the boreholes and test pits.

6.6.1 Conventional Spread and Strip Footing Foundations

As stated above, based on the ground surface elevations recorded at the borehole and test pit locations, it is anticipated that conventional foundations will provide a suitable foundation system for the central to northern sector of the property within the planned development. The foundations will be founded on the silty clay till, sandy silt till, silty clay, or compacted Engineered Fill.

Spread and strip footings founded on the underlying silty clay till, sandy silt till, or silty clay may be designed using values provided in Table 6.2.

Table 6.2: Footing on Native Silty Clay Till, Sandy Silt Till and Silty Clay

Footing Size	Factored ULS Resistance kPa	SLS Reaction kPa
Spread Footings		
1.2 m x 1.2 m	380	250
1.8 m x 1.8 m	400	190
2.4 m x 2.4 m	420	155
3.0 m x 3.0 m	450	135
Strip Footings		
0.45 m wide	220	220
0.6 m wide	240	230

The ULS values provided above include a resistance factor of 0.5. The SLS values have been calculated to provide a total settlement of 25 mm (or less) and differential settlement of 20 mm (¾ of the total settlement specified above), consistent with industry standard analysis.

The calculated values provided above for the foundations are based on a minimum of 1.2 m of soil required for adequate frost protection, in accordance with the freezing depths shown on OPSD Drawing 3090.101.

Where construction is undertaken during winter conditions, the footing sub-grade must be protected from freezing.

6.6.2 Trench Footings

It is common industry practice in southern Ontario to use trench footings for commercial/retail development where this approach is practical. This type of footing involves the excavation of the soil to the required depth, and the placement of concrete to fill the full width and depth of the excavation/trench. In this respect, the side walls of the excavation are used as the form work for the concrete footings.

Trench footings are therefore generally feasible where near-vertical side walls can be maintained in the excavations, and groundwater is absent or can be controlled and the excavation kept dry at the time of placement of the concrete.

Some of the overlying fill materials encountered in the boreholes and test pits are considered to be cohesionless materials. In this respect, this material will likely undergo sloughing and caving if cut vertically. It is also noted that some of the fill materials contains a high percentage of sand and silt, and that groundwater is present in this stratum within a depth of approximately 0.5 to 4.25 m below the existing ground surface.

Based the above, we consider trench footings are not feasible application on this Site.

6.6.3 Straight Shaft Drilled Piers (Caissons)

As a general guideline based on the conditions encountered in borehole BH10, caissons with a minimum length of 10.0 m founded with a minimum 1 m penetration into the very stiff to hard silty clay may be designed for the resistance at ULS and reaction at SLS provided in Table 6.3 below. The corresponding depth below existing grade and the recommended maximum founding elevation are also provided.

Table 6.3: Recommended Geotechnical Bearing Resistance and Reaction for Caisson Foundations

Infrastructure Component	BH No.	Anticipated Bearing Stratum	Bearing Resistance at U.L.S. (kPa)	Bearing Reaction at S.L.S. (kPa)	Maximum Founding Elevation (m)
Building	BH10	Very Stiff to Hard silty clay	1000	800	268.7 m to 263.4 m

The ULS value provided above include a resistance factor of 0.4, and the SLS value have been calculated to provide a total settlement of 25 mm (or less).

As indicated in Section 5.15, groundwater was encountered approximately from 4.25 m BGS to 0.5 m BGS; with flowing artesian conditions recorded. The use of caissons should be carefully considered, as the artesian conditions may require dewatering or depressurization of the artesian conditions using deep wells.

6.6.4 Pile Foundations

As an alternative to deep foundations consisting of caissons, steel piles driven with a minimum length of 10.0 m founded into the very stiff to hard silty clay may be designed as a general guideline based on the conditions encountered in borehole BH10 to support the building foundations.

Should pile foundations be considered an option, pile design analysis can be completed.

The capacities of the piles must be confirmed in the field using a Pile Driving Analyzer (PDA) to validate the design values used.

6.6.5 Alternative Foundation Considerations

Alternative foundation options using specialty foundation contractors may employed in the areas of deep earth fill. Two options that maybe explored are rammed aggregate piers and dynamic compaction. A brief discussion of the alternatives is noted below:

Rammed Aggregate Piers

As an alternative to deep foundations, ground improvement techniques could be utilized to support the foundations of the buildings using rammed aggregate piers in the areas of deep earth fill, i.e. – 3.4 m to 5.2 m thick.

The proposed building structures can be supported by strip/spread footings and standard slab-on-grade floors slab on existing soil reinforced by rammed aggregate piers. This technique results in a reinforced soil profile with less compressibility than the existing soil.

As a preliminary guideline bearing improvements in the order of up to 150 kPa at SLS and 225 kPa at ULS at the project site can be anticipated. The design and installation of the rammed aggregate piers is typically performed by a specialty contractor.

As indicated in Section 5.15, groundwater was encountered approximately from 4.25 m BGS to 0.5 m BGS; with flowing artesian conditions recorded. The use of rammed aggregate piers should be carefully considered as the artesian condition may require dewatering or depressurization of the artesian conditions using deep wells.

Dynamic Compaction

Dynamic compaction is a method that is used to increase the density in the areas of deep earth fill, i.e. – 3.4 m to 5.2 m thick.

The process of dynamic compaction involves of dropping a heavy weight repeatedly on the ground at regularly spaced intervals. The weight and the height determine the amount of compaction that would occur. The weight that is used, depends on the degree of compaction desired and is between 8 tonne to 36 tonne. The height varies from 1m to 30 m.

The degree of compaction depends on the weight of the hammer, the height from which the hammer is dropped, and the spacing of the locations at which the hammer is dropped. The initial weight dropping has the most impact and penetrates up to a greater depth. The following drops, if spaced closer to one another, compact the shallower layers and the process is completed by compacting the soil at the surface.

The design and installation of dynamic compaction method is typically performed by a specialty contractor.

The disadvantage of these alternative foundation methods on Site is the presence of buried topsoil and organic deposit. These deposits must be subexcavated and replaced with inorganic materials prior to using these alternative methods.

The required dynamic compaction method will generate some vibrations that will be perceptible to the nearby structures. Should there be structures in the area sensitive to vibrations, a maximum peak particle velocity specification should be developed by a vibration specialist. Pre-construction surveys of surrounding structures should be carried out. Vibration monitoring should be carried out prior to and throughout the construction period. Local by-laws should be confirmed and may include specific restrictions and/or monitoring requirements.

6.7 EARTHQUAKE DESIGN CONSIDERATIONS

The selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy. The deepest borehole BH7 advanced for the current investigation was terminated at 31.9 m below existing grade.

For the purposes of this report, the weighted average N-value method has been used to assess the Seismic Site Classification consistent with the second of three methods stated in the Ontario Building Code (2012).

Based on the comments provided above, the interpreted soil profile (conservatively taken to assume the presence of the fine grained – cohesive silty clay soils within the depth of interest) used to assess the Seismic Site Classification is as follows.

- Layer 1: Thickness = 1.5 m Average SPT N = 9 (Earth Fill)
- Layer 2: Thickness = 0.9 m Average SPT N = 2 (Topsoil)
- Layer 3: Thickness = 1.4 m Average SPT N = 4 (Alluvial Deposit)
- Layer 4: Thickness = 0.9 m Average SPT N = 3 (Silty Clay)
- Layer 5: Thickness = 0.8 m Average SPT N = 5 (Silty Sand)
- Layer 6: Thickness = 6.7 m Average SPTN = 10 (Silty Clay)
- Layer 7: Thickness = 3.0 m Average SPTN = 16 (Silty Sand)
- Layer 8: Thickness = 1.6 m Average SPTN = 13 (Silt)
- Layer 9: Thickness = 3.0 m Average SPTN = 16 (Silty Clay Till)
- Layer 10: Thickness = 1.3 m Average SPTN = 11 (Silty Sand Till)
- Layer 11: Thickness = 2.3 m Average SPTN = 18 (Sand)
- Layer 12: Thickness = 3.2 m Average SPTN = 9 (Silty Clay)
- Layer 13: Thickness = 5.1 m Average SPTN = 45 (Sand)

Based on the stratigraphy and N-values referenced a weighted harmonic average N-value of 13 was calculated. Based on this value, the site classification for seismic site response is Site Class E in accordance with Table 4.1.8.4.A of the 2012 Ontario Building Code (OBC).

The weighted harmonic average N-value calculated is considered low with respect to the subsurface conditions encountered in the region, and similar calculations for other projects.

Completion of a shallow seismic geophysics program to determine the shear wave velocity of the subsurface materials for comparison with the data in the table in the OBC would likely be the most expedient method of confirming and refining the Site Classification provided above.

6.8 FLOOR SLAB

Generally, the existing fill will require sub-excavation and replacement with approved fill consistent with the recommendations provided above for conventional slab-on-grade floor. Should the existing fill remain in place for building construction, a structural slab should be considered in areas where deep fill was encountered.

6.9 TEMPORARY EXCAVATIONS

Temporary excavations for the installation of buried services and utilities or for similar applications must be carried out in accordance with the latest edition of the Occupational Health & Safety Act (OH&S Act) & Regulations.

The alluvial deposit materials should be classified as Type 4 soils. In accordance with the OH&S Act, the maximum excavation side slope for a Type 4 soil is 3:1 (Horizontal: Vertical) extending from the base of the excavation.

The silty clay, sandy silt, silty sand, and sand and gravel fill materials should be classified as Type 3 soils. In accordance with the OH&S Act, the maximum excavation side slope for a Type 3 soil is 1:1 (Horizontal: Vertical) extending from the base of the excavation.

The native soils should be classified as Type 2 soils. In accordance with the OH&S Act, the maximum excavation side slope for a Type 2 soil is 1:1 (Horizontal: Vertical) but a vertical cut of 1.2 m is permitted extending from the base of the excavation.

Excavations that extend below the groundwater table are classified as Type 4 soils. The maximum side slope for Type 4 soils is 3:1 (Horizontal: Vertical) from the base of the excavation.

For the purpose of this report and the comments provided above, we have presumed that temporary excavations without lateral support (such as required for service and utility trenches) will remain open for relatively short periods (e.g. Typically 48 hours to 72 hours).

For excavations without lateral support that remain open for longer periods (such as for open excavations for demolition or for construction of underground storage tanks) sloughing and caving of side slopes must be anticipated, particularly if left exposed to periods of precipitation, ground surface runoff, or freeze-thaw cycles. The slopes of these excavations should be protected from erosion and the slopes should be inspected frequently for signs or indications of erosion and/or instability.

If localized instability is noted at the time of excavation or while an excavation remains open, or if wet conditions are encountered, the side slopes of the excavations should be flattened as required to maintain safe working conditions.

Stockpiling of any materials adjacent to excavations should be avoided. Similarly, traffic should not be permitted in proximity to open excavations. For this purpose, it is recommended that all storage of materials and traffic be restricted from a 3 m wide strip around the excavations, measured from the crest of the excavation designed and constructed in accordance with the OH&S Act.

If space is restricted such that the side slope cannot be safely cut back in accordance with the OH&S Act & Regulations, if sloughing and cave-in are encountered in the excavations, or if the excavations are to remain open for a longer period, a trench box system can be used for shallow localized excavations (such as for service trenches), or a shoring system can be used for larger or deeper excavations (building excavations), to maintain safe working conditions.

6.10 UNDERGROUND STORAGE TANKS

The conceptual design, location and specification of the storage tanks for the potential gas station is in progress, but not complete at the time of finalizing this report.

Typically, a rigid foundation such as concrete slab founded on undisturbed soils may be used to support the storage tanks. Based on the conditions encountered in borehole BH7 advanced in the area of the tanks and assuming a founding level of 4 m below grade, it is anticipated that the excavation for the tanks will terminate on the soft to firm silty clay soils. These native soils will likely provide a suitable base and bearing resistance for the installation of small-size tanks (up to 3 m to 4 m high).

As indicated in Section 5.15, groundwater was encountered approximately from 4.25 m BGS to 0.5 m BGS; with flowing artesian conditions recorded. Shallow groundwater conditions may require dewatering or depressurization of the shallow groundwater system to complete installation of the storage tank foundation(s).

6.11 SEPTIC TILE BED PARAMETER

For general reference, the results of the grain size distribution tests (and Unified Soil Classifications) completed on the predominant soil strata encountered in the boreholes and test pits has been compared to the grain size curves and soil types referenced in Supplementary Standard SB-6 of the 2012 Ontario Building Code Compendium. This reference can be used as a guideline to estimate the likely range in the coefficient of permeability of the soils encountered in the investigation. It is noted that the industry typically refers to "hydraulic conductivity" rather than "coefficient of permeability" in this respect. The terms are often considered interchangeable, but for purposes of this report the values provided are in the form of "length/time" (cm/sec) and are therefore considered strictly applicable to "hydraulic conductivity", and hence "hydraulic conductivity" is used herein.

As presented in Section 5.13, analytical results indicate that the horizontal hydraulic conductivity for the sand to silty clay material present within the well screened intervals ranges from 1.0×10^{-8} m/s to 5.9×10^{-6} m/s (1.0×10^{-6} cm/s to 5.9×10^{-4} cm/s), with a geomean of 4.3×10^{-7} m/s (4.3×10^{-5} cm/s).

The Supplementary Standard states in part that "it must be emphasized that, particularly for fine grained soils, there is no consistent relationship (between coefficient of permeability and soils of various types) due to the many factors involved". Such factors as structure, mineralogy, density (compactness or consistency), plasticity, and organic content of the soil can have a large influence on the hydraulic conductivity; variations in excess of an "order of magnitude" are common place in this respect.

Given the fine-grained nature of the native soils together with the shallow groundwater table conditions across most of the site, it is likely that raised septic beds will be required.

6.12 ACCESS ROAD AND PARKING AREAS

Provided that the exposed sub-grade surface is prepared in accordance with the recommendations provided in the previous sections of this report, and all required earthworks are conducted as recommended herein, the asphalt pavement structures provided below can be considered for use at this site.

Table 6.4: Pavement

Material	Standard Duty Parking Areas	Heavy Duty Fire and Truck Routes	Compaction Requirements
HL3 (top course asphaltic concrete)	40 mm	40 mm	97 % BRD
HL8 (base course asphaltic concrete)	50 mm	60 mm	97 % BRD
OPSS Granular 'A' Base or 19mm recycled material	150 mm	150 mm	100 % SPMDD
OPSS Granular 'B' Sub-base or 50mm recycled material	200 mm	350 mm	100 % SPMDD

The designs shown above should provide a pavement service life in the order of 15 years (which is considered typical for commercial developments), although additional operation and maintenance effort beyond that considered typical, may be required during the life cycle of the pavements.

The base and sub-base materials should be compacted to a minimum of 100% SPMDD. The asphaltic concrete should be compacted to a minimum of 97% Bulk Relative Density (BRD).

The finished sub-grade surface and the pavement (asphalt and concrete) surface should be crowned and graded to direct runoff water away from the building, sidewalks, roadways, and associated infrastructure.

7.0 CLOSURE

Use of this report is subject to the Statement of General Conditions in Appendix A. It is the responsibility of S. Larkin Developments Inc., who is identified as "the Client" in the Statement of General Conditions, and its agents, to review the conditions and to notify Stantec should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report
- Basis of the report
- Standard of care
- Interpretation of site conditions
- Varying of unexpected site conditions
- Planning, design or construction

Respectfully submitted,

STANTEC CONSULTING LTD.

Eric Fron, C.E.T.
Senior Geotechnical Consultant
eric.fron@stantec.com

Roger Freymond, B.A.Sc., P.Eng.
Senior Hydrogeologist
roger.freymond@stantec.com

Ron Howieson, P.Eng.
Senior Principal, Geotechnical Engineering
ron.howieson@stantec.com

PROFESSIONAL ENGINEER
LICENSED
Jan. 17 2019
R.W. HOWIESON
Geotechnical
PROVINCE OF ONTARIO

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APPENDIX A

Statement of General Conditions

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

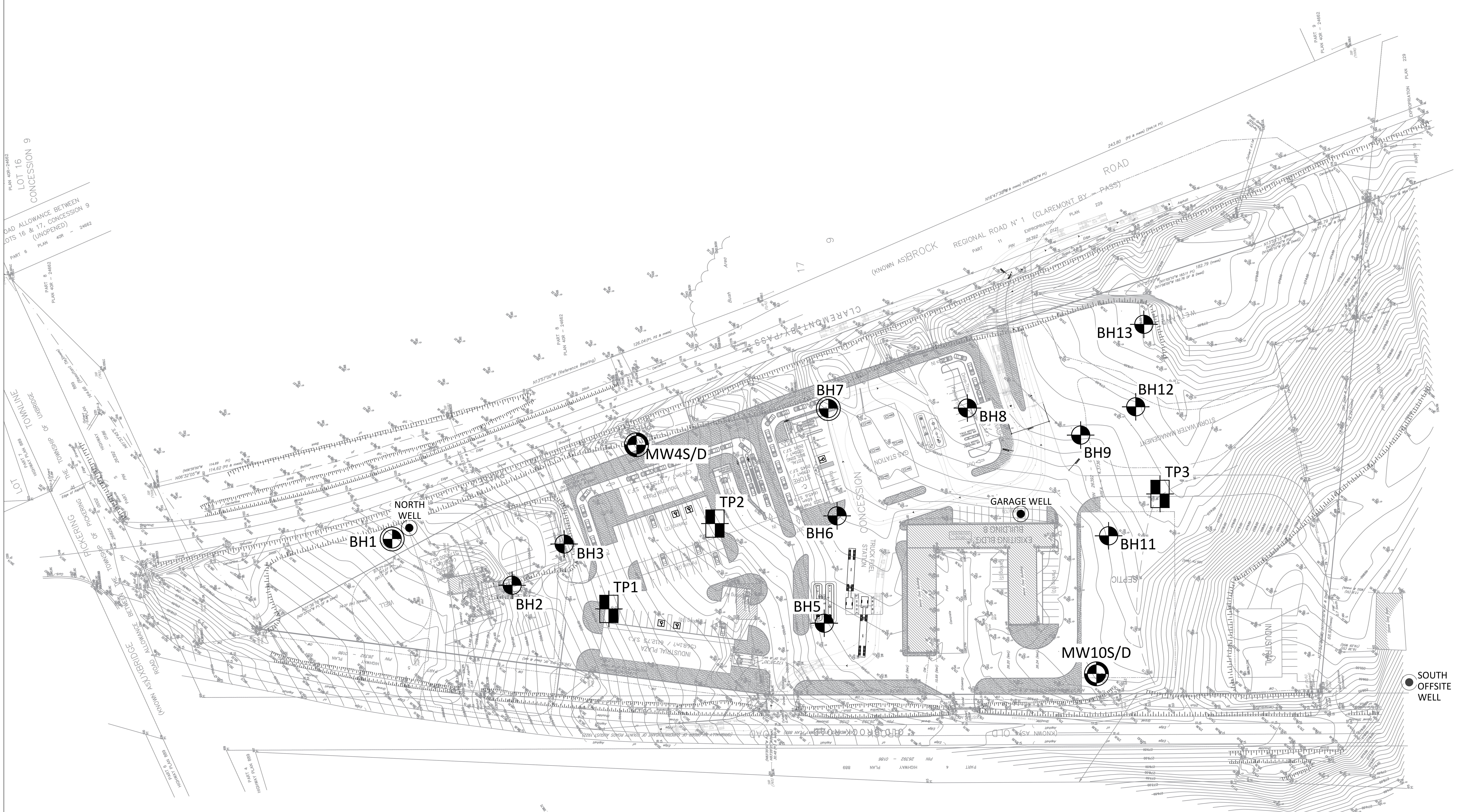
STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

APPENDIX B



ORIGINAL SHEET - ARCH D

Stantec

300 - 675 Cochrane Drive West Tower
Markham, Ontario L3R 0B8
Tel. 905.944.7777
www.stantec.com

Consultants

Legend

TEST PIT LOCATION

BOREHOLE LOCATION

PROPOSED BOREHOLE (not drilled during current preliminary investigation)

PRIVATE WELL LOCATION

Notes

METRIC

DISTANCES AND COORDINATES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048.

DISTANCES SHOWN ON THIS PLAN ARE GROUND DISTANCES AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.99983990.
DIGITAL DRAWING HAS BEEN PREPARED IN GRID.

BEARINGS SHOWN ON THIS PLAN ARE UTM GRID BEARINGS AND ARE DERIVED FROM SPECIFIED CONTROL POINTS # 01019890825, # 00819830450 AND # 00819760184 , UTM ZONE 17, NAD 83 CSRS (2010.0 EPOCH).

SPECIFIED CONTROL POINTS (SCP's) AND OBSERVED REFERENCE POINTS (ORP'S) UTM ZONE 17, NAD83 CSRS (2010.0 EPOCH) COORDINATES TO RURAL ACCURACY PER SEC.14 (2) OF O.REG. 216/10.

POINT ID	NORTHING	EASTING
SCP # 01019890825	4874901.478	656961.700
SCP # 00819830450	4864297.481	651249.915
SCP # 00819760184	4874971.290	636891.014
ORP "A"	4872199.1	649882.7
ORP "B"	4872598.6	649944.3
ORP "C"	4872164.0	650067.1

COORDINATES CANNOT, IN THEMSELVES, BE USED TO RE-ESTABLISH CORNERS OF BOUNDARIES SHOWN ON THIS PLAN.

Revision	By	Appd.	YY.MM.DD

Issued	By	Appd.	YY.MM.DD

File Name:	Dwn.	Chkd.	Dsgn.	YY.MM.DD

Permit-Seal

Client/Project

S. Larkin Developments Inc.
Claremont North Business Park

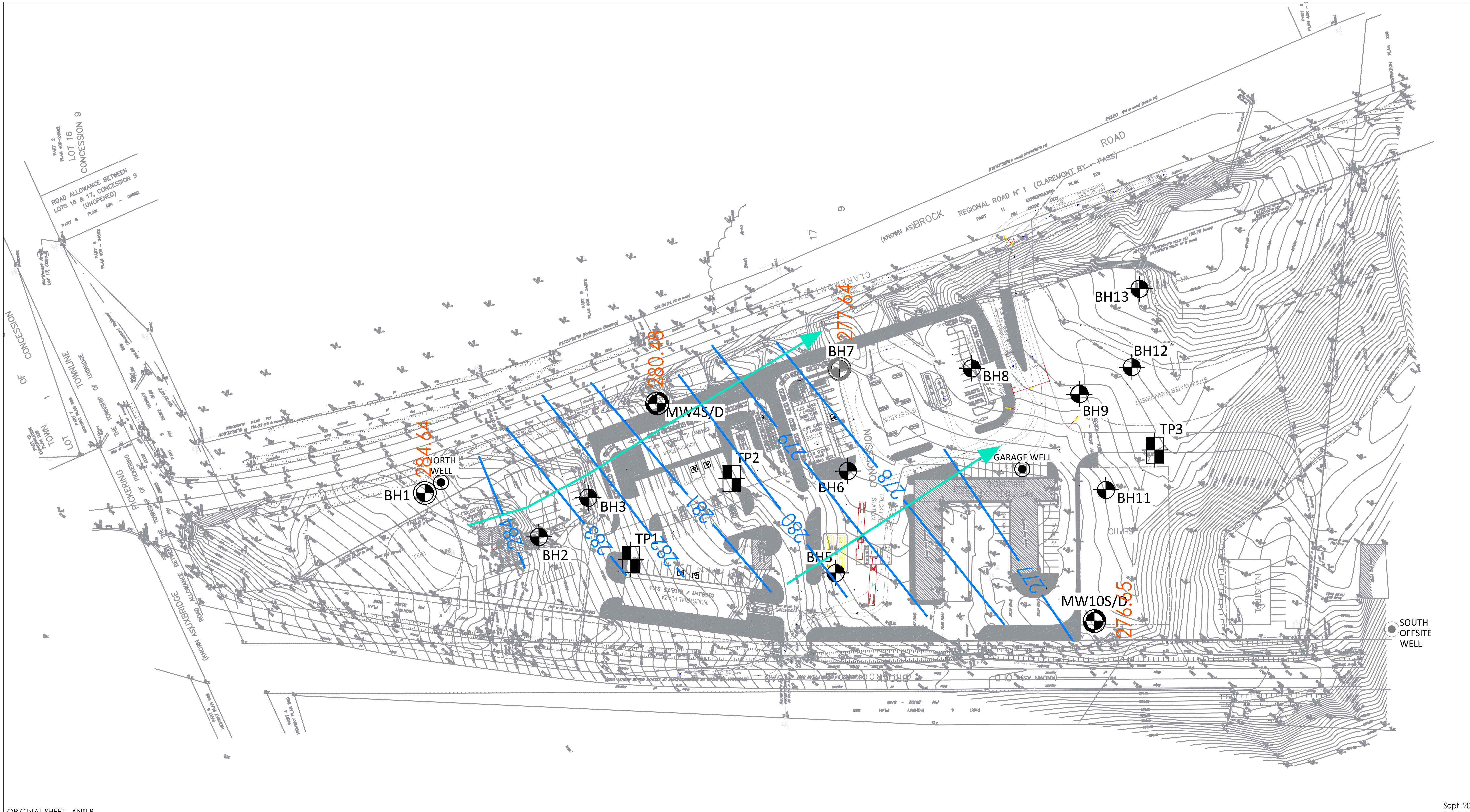
Pickering, ON

Title

Borehole Location Plan

Project No.	Scale
160622415	1:750

Drawing No.	Sheet	Revision
1	of	



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2017/09/15 1:31 PM By: Van Damme, Steven

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Legend

- BOREHOLE
- TEST PIT LOCATION
- PROPOSED BOREHOLE (not drilled during current preliminary investigation)
- PRIVATE WELL LOCATION
- INFERRED DIRECTION OF GROUNDWATER FLOW
- SHALLOW GROUNDWATER CONTOUR
- 279 GROUNDWATER ELEVATION

Notes

1:1500 — 0 15 45 75m

Client/Project
S. Larkin Developments Inc.
Claremont North Business Park

Figure No.
2

Title
SHALLOW GROUNDWATER CONTOURS

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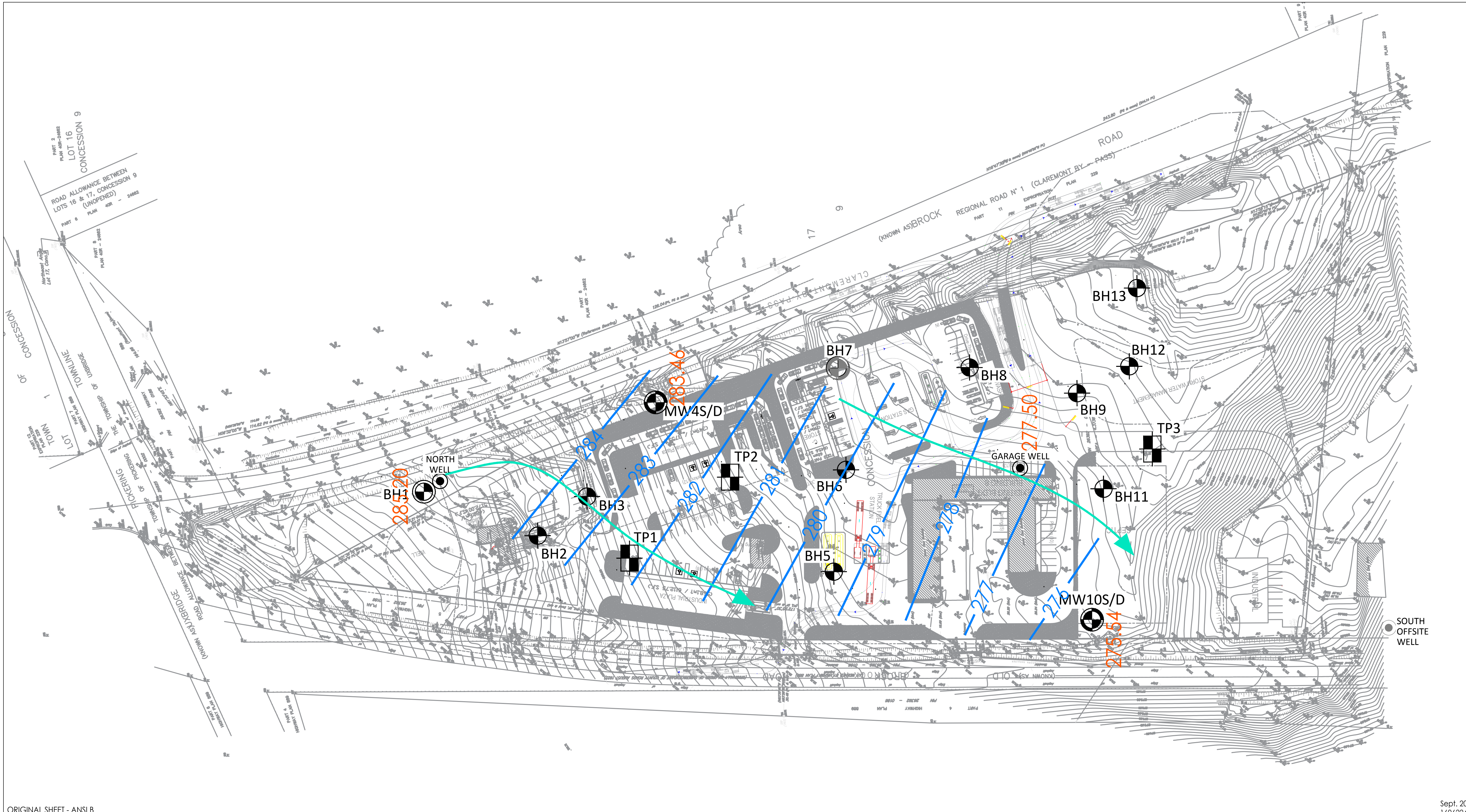
2017/09/15 1:30 PM By: Van Damme, Steven



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Legend

- BOREHOLE
- TEST PIT LOCATION
- PROPOSED BOREHOLE (not drilled during current preliminary investigation)
- PRIVATE WELL LOCATION
- INFERRED DIRECTION OF GROUNDWATER FLOW
- DEEP GROUNDWATER CONTOUR
- 279 GROUNDWATER ELEVATION

Notes

1:1500 0 15 45 75m

Client/Project
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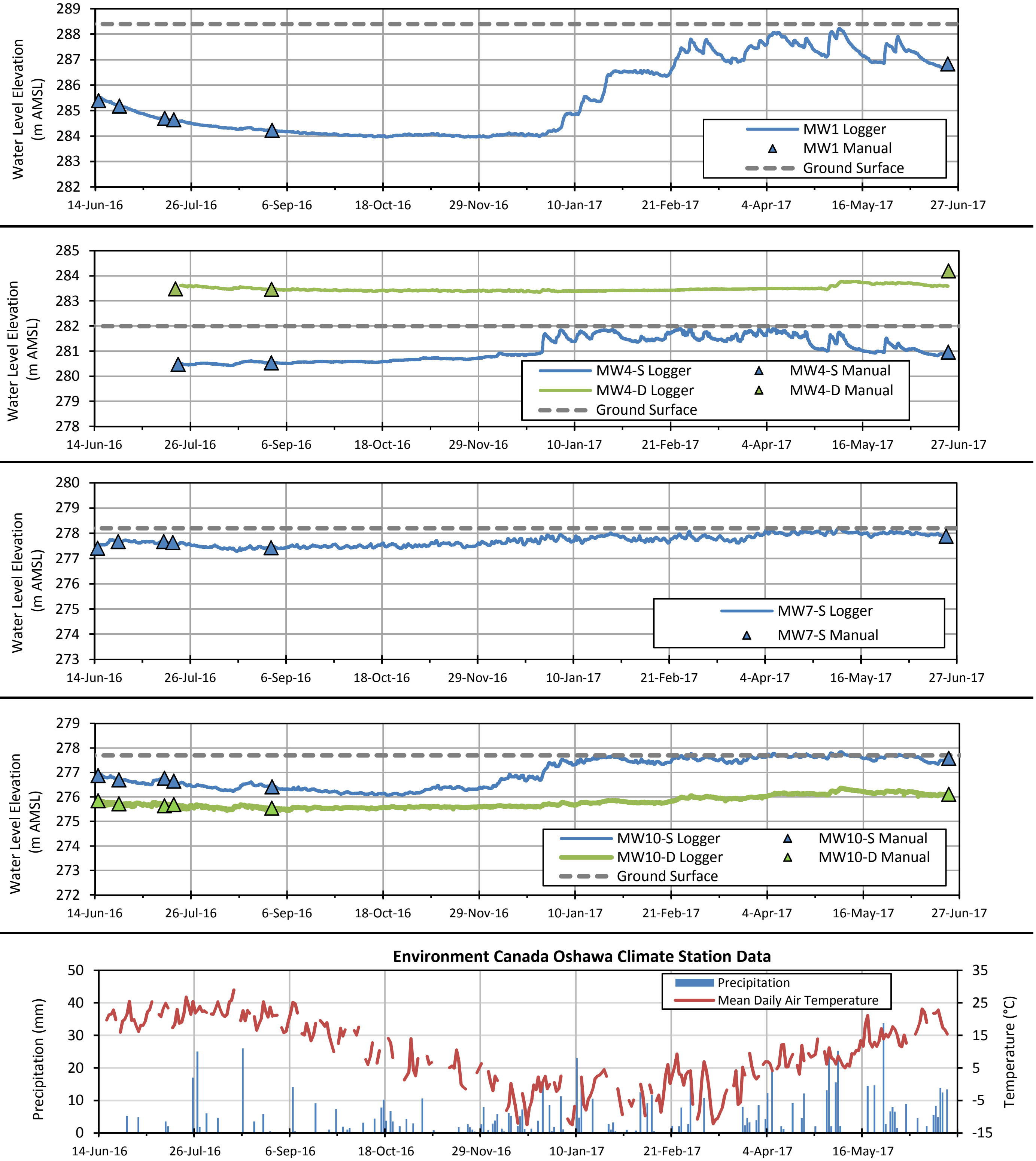
Claremont North Business Park

Figure No.
3

Title
DEEP GROUNDWATER CONTOURS



Notes:

Climate data obtained from Environment Canada Oshawa Climate Station and supplemented with data from Uxbridge Climate Station.

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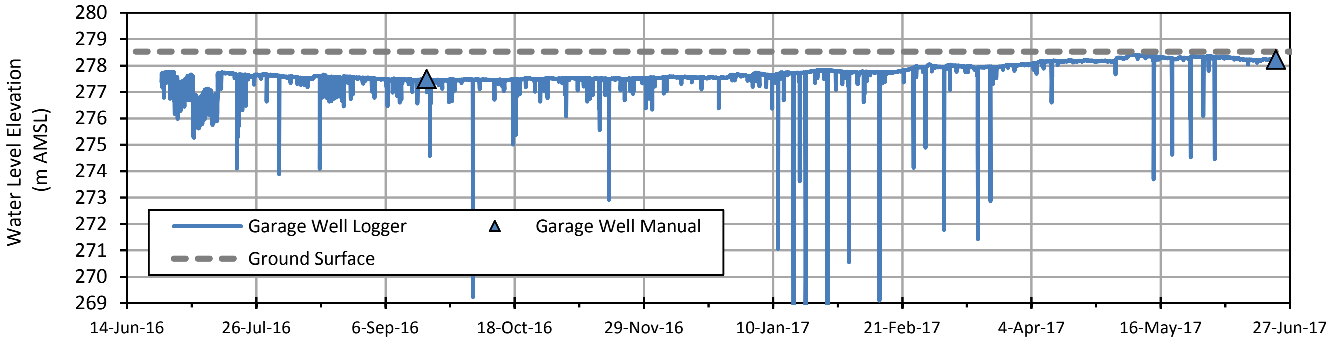
Geotechnical / Hydrogeological Report
Claremont Business Park
S. Larkin Developments

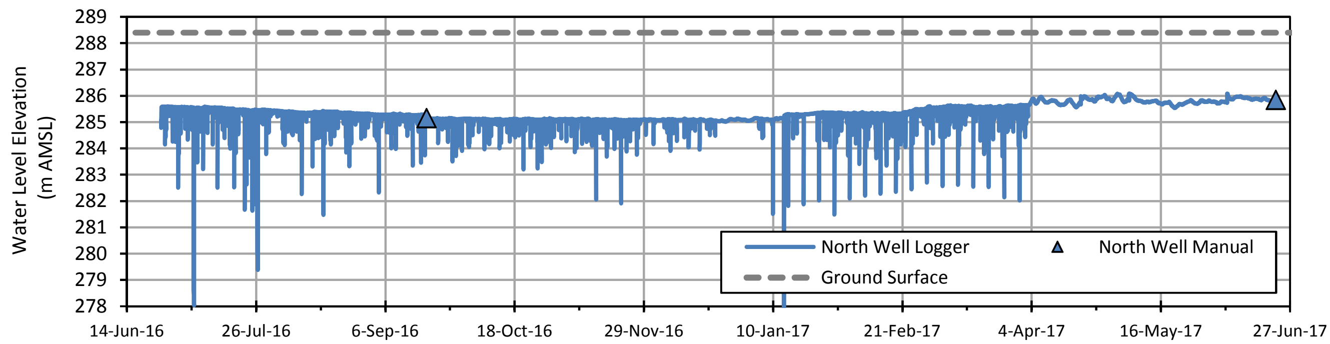
Figure No.

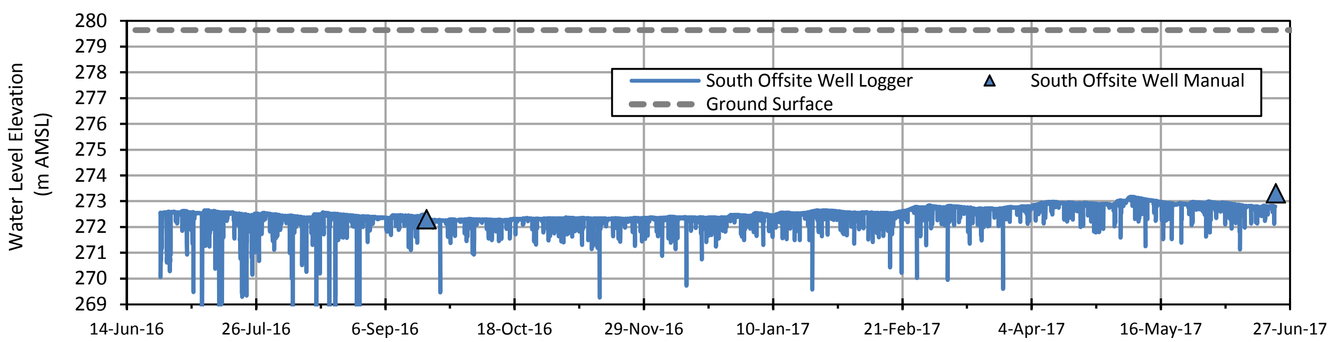
4

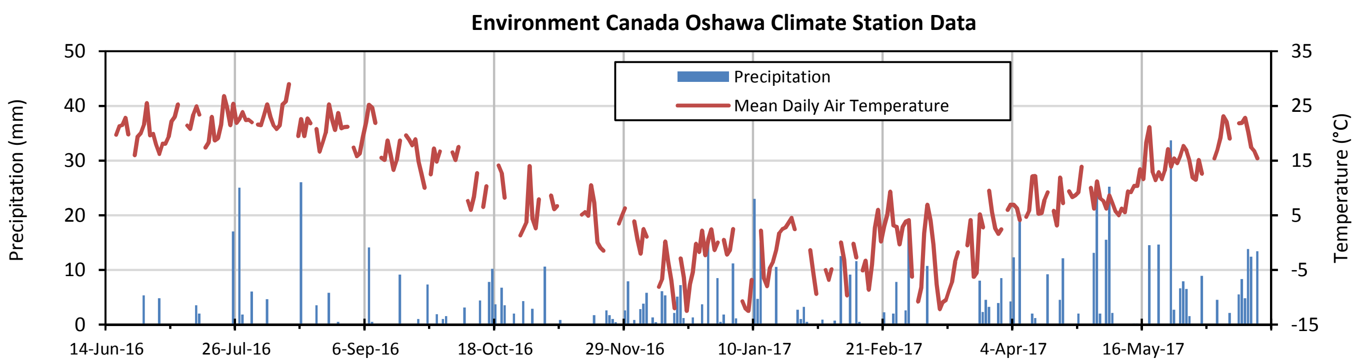
Title

Monitoring Well Hydrographs









Notes:

Climate data obtained from Environment Canada Oshawa Climate Station and supplemented with data from Uxbridge Climate Station.

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Client/Project

Geotechnical / Hydrogeological Report
Claremont Business Park
S. Larkin Developments

Figure No.

5

Title

Private Well Hydrographs

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Appendix B - Figure 6
Well Construction Details
Claremont Business Park

Monitor ID	Date Completed	Location: Northing NAD83	Location: Easting NAD83	Elevations: Ground Surface (mAMSL)	Elevations: Top of Pipe (mAMSL)	Borehole Information: Diameter (mm)	Borehole Information: Total Depth (mBGS)	Well Casing: Depth (mBGS)	Well Casing: Diameter (mm)	Well Casing: Measured Stick-up (mAGS)	Screened Interval: Top (mbTOP)	Screened Interval: Top (mAMSL)	Screened Interval: Bottom (mbTOP)	Screened Interval: Bottom (mAMSL)	Screened Material: Description	Estimated Hydrualic Conductivity (m/s)
Monitoring Wells																
MW1	Jun-16	4872545	649951	288.40	289.14	229	5.18	5.015	51	0.74	2.71	286.43	5.76	283.39	Silty Clay to Silty Sand Till	3.8E-07
MW7-S	Jun-16	4872379	650000	278.20	278.94	229	4.88	4.75	51	0.74	2.44	276.50	5.49	273.45	Silty Clay to Silty Sandy Clay	9.9E-07
MW4-S	Jul-16	4872452	649987	282.00	282.85	229	4.88	4.57	51	0.85	2.37	280.48	5.42	277.43	Clayey Silty Sand to Silty Clay	1.0E-08
MW4-D	Jul-16	4872452	649987	282.00	282.80	229	29.26	28.96	38	0.80	26.71	256.09	29.76	253.04	Sand	5.0E-06
MW10-S	Jun-16	4872278	649900	277.70	278.45	229	4.68	4.68	51	0.75	2.38	276.07	5.43	273.02	Clayey Silt to Silty Clay	5.9E-08
MW10-D	Jun-16	4872278	649900	277.70	278.61	229	30.18	25.23	51	0.91	23.09	255.52	26.14	252.47	Silty Clay to Silty Clayey Sand Till	5.9E-06

Notes:
n/a = not available
mm = millimetres
m = metres
mAMSL = metres above mean sea level
mBGS = metres below ground surface
mAGS = metres above ground surface

Appendix B - Figure 7
Summary of Groundwater Analytical Results - PW-01
Claremont Business Park

Sample Location			Claremont Business Park				
Sample Date			15-Jun-16	15-Jun-16	15-Jun-16	15-Jun-16	15-Jun-16
Sample ID			MW1	MW7-S	MW7-D	MW10-S	MW10-D
Water Type			Raw	Raw	Raw	Raw	Raw
Sample Tap			Waterra	Waterra	Waterra	Waterra	Waterra
Treatment Type			None	None	None	None	None
Sampling Company			STANTEC	STANTEC	STANTEC	STANTEC	STANTEC
Laboratory			MAXX	MAXX	MAXX	MAXX	MAXX
Laboratory Work Order			B6C4128	B6C4128	B6C4128	B6C4128	B6C4128
Laboratory Sample ID			CNY527	CNY528	CNY529	CNY530	CNY531
Filtered	Units	ODWS	Total Metals	Total Metals	Total Metals	Total Metals	Total Metals
General Chemistry							
Alkalinity, Bicarbonate (as CaCO3)	mg/L	n/v	290	580	610	770	420
Alkalinity, Carbonate (as CaCO3)	mg/L	n/v	1.6	1.1	1.0	2.0	1.7
Alkalinity, Total (as CaCO3)	mg/L	30-500^{E}	290	**580**	**610**	**770**	420
Ammonia (as N)	mg/L	n/v	<0.050	8.5	8.8	6.7	3.5
Anion Sum	meq/L	n/v	8.83	12.6	13.2	34.3	18.7
Cation Sum	meq/L	n/v	9.14	15.2	14.2	35.9	22.4
Chloride (Dissolved)	mg/L	250^{D}	67	34	33	**670**	**330**
Dissolved Organic Carbon (DOC)	mg/L	5^{D}	**5.3**	**20**	**20**	**16**	**7.2**
Electrical Conductivity, Lab	µmhos/cm	n/v	860	1200	1200	3500	2000
Hardness (as CaCO3)	mg/L	80-100^{E}	**390**	**620**	**580**	**1000**	**500**
Ion Balance	%	n/v	1.74	9.39	3.58	2.31	9.03
Langelier Index (at 20 C)	none	n/v	0.781	0.860	0.771	1.21	0.896
Langelier Index (at 4 C)	none	n/v	0.533	0.613	0.524	0.970	0.650
Nitrate (as N)	mg/L	10.0_{d}^{B}	0.83	<0.10	<0.10	<0.10	0.12
Nitrite (as N)	mg/L	1.0_{d}^{B}	0.013	0.011	0.010	<0.010	0.025
Orthophosphate(as P)	mg/L	n/v	<0.010	<0.010	<0.010	<0.010	<0.010
pH	S.U.	6.5-8.5^{E}	7.75	7.32	7.24	7.44	7.63
Saturation pH (at 20 C)	none	n/v	6.97	6.46	6.47	6.23	6.73
Saturation pH (at 4 C)	none	n/v	7.22	6.71	6.72	6.47	6.98
Sulfate (Dissolved)	mg/L	500_{h}^{D}	48	<1.0	<1.0	<1.0	44
Total Dissolved Solids	mg/L	500^{D}	480	**710**	**700**	**1900**	**1100**
Metals							
Aluminum	µg/L	100^{E}	**450**	**470**	**340**	**140**	**420**
Antimony	µg/L	6^{C}	1.9	0.95	0.62	<0.50	<0.50
Arsenic	µg/L	25^{C}	<1.0	3.6	3.0	<1.0	2.3
Barium	µg/L	1000^{B}	130	320	290	510	210
Beryllium	µg/L	n/v	<0.50	<0.50	<0.50	<0.50	<0.50
Boron	µg/L	5000^{C}	19	35	34	13	22
Cadmium	µg/L	5^{B}	<0.10	0.17	0.14	0.17	<0.10
Calcium	µg/L	n/v	100000	190000	180000	340000	160000
Chromium (Total)	µg/L	50^{B}	<5.0	<5.0	<5.0	<5.0	<5.0
Cobalt	µg/L	n/v	2.2	44	43	16	2.1
Copper	µg/L	1000^{D}	5.1	7.7	5.0	6.6	1.9
Iron	µg/L	300^{D}	**860**	**13000**	**9600**	**8000**	**9000**
Lead	µg/L	10_{c}^{B}	<0.50	3.3	2.2	<0.50	2.2
Magnesium	µg/L	n/v	32000	34000	33000	48000	25000
Manganese	µg/L	50^{D}	**150**	**3000**	**3100**	**2600**	**600**
Molybdenum	µg/L	n/v	2.6	2.3	2.4	1.7	8.0
Nickel	µg/L	n/v	2.9	9.5	8.7	6.0	2.7
Phosphorus	µg/L	n/v	<100	300	230	<100	<100
Potassium	µg/L	n/v	4900	8700	8300	5500	6300
Selenium	µg/L	10^{B}	<2.0	<2.0	<2.0	<2.0	<2.0
Silicon	µg/L	n/v	6500	14000	13000	11000	10000
Silver	µg/L	n/v	<0.10	<0.10	<0.10	<0.10	<0.10
Sodium	µg/L	200000_{a}^{D} 20000_{a}^{F}	**24000**	**32000**	**33000**	**330000**	**260000**
Strontium	µg/L	n/v	440	750	700	1100	580
Thallium	µg/L	n/v	<0.050	0.068	0.067	0.061	<0.050
Titanium	µg/L	n/v	40	16	13	8.2	15
Uranium	µg/L	20^{B}	10	5.8	5.7	3.7	4.7
Vanadium	µg/L	n/v	1.5	2.2	1.7	0.81	1.2
Zinc	µg/L	5000^{D}	7.9	36	10	45	9.4

Appendix B - Figure 7
Summary of Groundwater Analytical Results - PW-01
Claremont Business Park

Notes:

ODWS	Technical Support Document for Ontario Drinking Water Standards, Objectives and Guidelines (MOE, 2006)
A	ODWS Table 1 - Microbiological Standards, Maximum Acceptable Concentration
B	ODWS Table 2 - Chemical Standards, Maximum Acceptable Concentration
C	ODWS Table 2 - Chemical Standards, Interim Maximum Acceptable Concentration
D	ODWS Table 4 - Chemical/Physical Objectives and Guidelines, Aesthetic Objectives
E	ODWS Table 4 - Chemical/Physical Objectives and Guidelines, Operational Guidelines
F	ODWS Table 4 - Medical Officer of Health Reporting Limit
6.5[A]	Concentration exceeds the indicated standard.
15.2	Concentration was detected but did not exceed applicable standards.
< 0.50	Laboratory reportable detection limit exceeded standard.
< 0.03	The analyte was not detected above the laboratory reportable detection limit.
n/v	No standard/guideline value.
-	Parameter not analyzed / not available.
b	Where fluoride is added to drinking water, it is recommended that the concentration be adjusted to 0.5 - 0.8 mg/L the optimum level for control of tooth decay. Where supplies contain naturally occurring fluoride at levels higher than 1.5 mg/L but lower than 2.4 mg/L the Ministry of Health and Long Term Care recommends an approach through local boards of health to raise public and professional awareness to control excessive exposure to fluoride from other sources.
c	This standard applies to water at the point of consumption. Since lead is a component in some plumbing systems, first flush water may contain higher concentrations of lead than water that has been flushed for five minutes.
d	Where both nitrate and nitrite are present, the total of the two should not exceed 10 mg/L (as nitrogen).
e	The standard is expressed as a running annual average of quarterly samples measured at a point reflecting the maximum residence time in the distribution system.
f	Refer to ODWS Table 2 for health related standard
g	The aesthetic objective for sodium in drinking water is 200 mg/L. The local Medical Officer of Health should be notified when the sodium concentration exceeds 20 mg/L so that this information may be communicated to local physicians for their use with patients on sodium restricted diets.
h	When sulfate levels exceed 500 mg/L, water may have a laxative effect on some people.
i	Applicable for all waters at the point of consumption.
j	The operational guidelines for filtration processes are provided as performance criteria in the Procedure for Disinfection of Drinking Water in Ontario.
s1	Standard is applicable to total xylenes, and m & p-xylenes and o-xylenes should be summed for comparison.
MI	Detection limit was raised due to matrix interferences.

APPENDIX C

Symbols and Terms Used on Borehole Records

Borehole Records

Stantec

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta-morphic Bedrock	Sedi-mentary Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼	measured in standpipe, piezometer, or well
▽	inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

BOREHOLE RECORD

Stantec

BH 1

CLIENT: S. Larkin Developments Inc.

PROJECT No.: 160622415

LOCATION: 5433 Old Brock Road, Pickering, ON

DATUM:

DATES: BORING June 13, 2016

WATER LEVEL: August 30, 2016

TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	WATER LEVEL	DEPTH (ft)	SAMPLE TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	288.4			0				
	288.2	230 mm TOPSOIL						
		FILL: brown, silty clay, some sand and gravel - occasional organic inclusions - moist to wet		1	SS	1	460/610	21
				3–4	SS	2	380/610	37
	286.9	Very stiff to hard, brown, silty CLAY (CL-ML), some sand - moist to wet		6	SS	3	480/610	28
				8–9	SS	4	460/610	29
3		- grey at 3.0 m		11	SS	5	510/610	19
			▼ (≈4.1 m)	13–14	SS	6	500/610	33
	283.2			16	SS	7	380/610	44
		END OF BOREHOLE at 5.2 m below existing grade. Groundwater level was measured in monitoring well, MW1, at approximately 4.1 m below existing grade on August 30, 2016.						

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

BH 4

Stantec

CLIENT: S. Larkin Developments Inc.
PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON
DATUM:
DATES: BORING: July 14 to July 18, 2016
WATER LEVEL: August 30, 2016
TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	282.0	100 mm TOPSOIL
		FILL: brown silty clay, trace gravel and sand - moist to wet
	280.5	Firm to stiff, grey, silty CLAY (CL-ML), - sand layers - moist to wet
	274.5	Very loose, grey, silty SAND (SM), TILL - trace gravel - trace clay - moist to wet
	273.0	Firm to stiff, grey, silty CLAY (CL-ML), - some sand, - moist to wet
12		Continued Next Page

Water level symbol (▼) shown at approx. 1.35 m depth (4.5 ft).

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
1	SS	1	300/610	6
3.5	SS	2	420/610	8
6	SS	3	430/610	9
8.5	SS	4	550/610	10
11	SS	5	520/610	11
13.5	SS	6	0.0/610	6
16	SS	7	530/610	7
18.5	SS	8	430/610	5
21	SS	9	230/610	7
26	SS	10	250/610	0
31	SS	11	370/610	6
36	SS	12	430/610	15

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼
STANDARD PENETRATION TEST, BLOWS/0.3m ●

Scale: 10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test, kPa
◘ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

BH 4

CLIENT S. Larkin Developments Inc.

PROJECT No. 160622415

LOCATION 5433 Old Brock Road, Pickering, ON

DATUM

DATES: BORING July 14 to July 18, 2016

WATER LEVEL August 30, 2016

TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	SAMPLE TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
12	270.0	Loose to dense, grey SAND (SP) - some silt - trace clay - moist	41	SS	13	200/610	4
14		- with occasional wet silt seams and layes	46	SS	14	300/610	13
15			51	SS	15	380/610	26
17			56	SS	16	510/610	34
18			61	SS	17	510/610	37
19.7	262.3	Firm to stiff, grey, silty CLAY (CL-ML) - trace to some sand - moist to wet	66	SS	18	410/610	15
21			71	SS	19	320/610	15
23			76	SS	20	390/610	4

UNDRAINED SHEAR STRENGTH (kPa) 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

□ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

Stantec

BH 4

CLIENT: S. Larkin Developments Inc.
PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON
DATUM:
DATES: BORING July 14 to July 18, 2016
WATER LEVEL: August 30, 2016
TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
24	258.0		79				
	257.8	Very loose to compact, grey fine SAND (SP) - some silt - trace to some clay - moist to wet	80–81	SS	21	200/610	0
25–26		- with occasional wet seams and layers	85–86	SS	22	380/610	24
27–28			90–91	SS	23	470/610	15
29	252.4		95–96	SS	24	250/610	15
		END OF BOREHOLE at 29.6 m below existing grade. Groundwater level was measured in monitoring well, MW4, at 1.4 m below existing grade on August 30, 2016.	97				

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P, W, W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

Stantec

BH 7

CLIENT: S. Larkin Developments Inc.
PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON
DATUM:
DATES: BORING June 8 to June 10, 2016
WATER LEVEL: August 30, 2016
TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	WATER LEVEL	SAMPLE TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0	278.2	100 mm TOPSOIL					
		FILL: dark brown silty clay with organic inclusions and layers - moist to wet	▼ (approx. 0.7 m)	SS	1	430/610	13
				SS	2	170/610	4
	276.7	Loose, dark brown, TOPSOIL - wet		SS	3	290/610	2
	275.8	Loose, dark brown, ALLUVIAL deposit with decayed vegetation - wet		SS	4	420/610	5
				SS	5	300/610	3
	274.4	Soft to firm, grey silty CLAY (CL) - trace sand - occasional wet sand and silt seams and layers - wet		SS	6	380/610	3
	273.5	Loose, grey, silty SAND (SM) - with occasional silty clay seams and layers		SS	7	430/610	5
	272.7	Firm to stiff, brown silty CLAY (CL-ML) - some sand - moist to wet		SS	8	610/610	5
				SS	9	610/610	10
				SS	10	530/610	14
				SS	11	610/610	10
				SS	12	510/610	10
10							

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UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P — W — W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

□ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec BOREHOLE RECORD

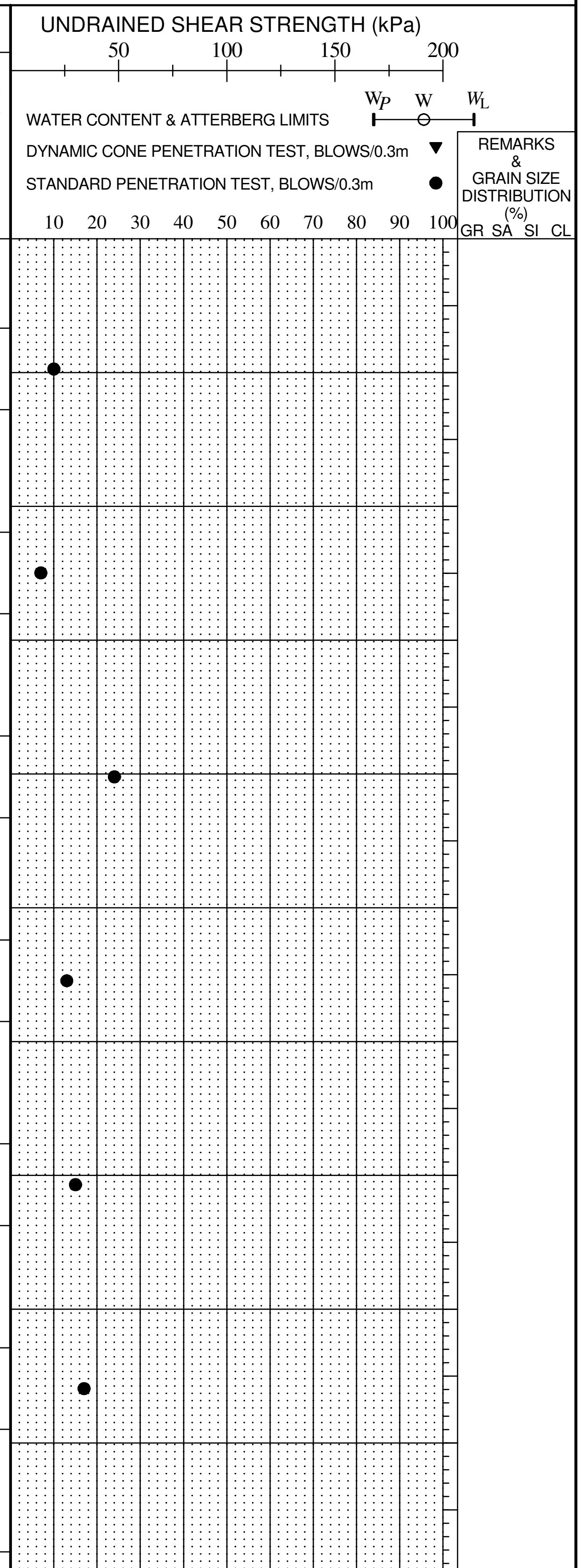
BH 7

CLIENT: S. Larkin Developments Inc. PROJECT No. 160622415

LOCATION: 5433 Old Brock Road, Pickering, ON DATUM

DATES: BORING June 8 to June 10, 2016 WATER LEVEL August 30, 2016 TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	SAMPLES TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
10	**268.2**		33–35				
		Stiff, grey, silty CLAY (CL-ML) - trace sand - moist to wet - occasional wet silt seams and layers	36	SS	13	430/610	10
12	266.0	Loose to compact, grey, silty SAND (SM) - trace gravel - moist to wet	41	SS	14	530/610	7
			46	SS	15	610/610	24
15	263.0	Compact, grey SILT (ML) - moist - some clay and sand - trace gravel	51	SS	16	380/610	13
	261.4	Stiff to very stiff, grey, silty CLAY (CL-ML), sandy, trace gravel, TILL - moist to wet	56	SS	17	530/610	15
			61	SS	18	410/610	17
	258.4		65				
20							



Continued Next Page

□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

Stantec

BH 7

CLIENT: S. Larkin Developments Inc. — PROJECT No.: 160622415

LOCATION: 5433 Old Brock Road, Pickering, ON — DATUM:

DATES: BORING: June 8 to June 10, 2016 — WATER LEVEL: August 30, 2016 — TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
20	258.2	Compact, grey, silty SAND (SM), trace gravel, TILL - moist
21.3	256.9	Compact, dark grey, fine SAND (SP) - some silt - moist to wet
23.6	254.6	Very soft to very stiff, grey silty CLAY (CL-ML) - some sand - moist - with occasional silt seams and layers
26.8	251.4	Compact to very dense, grey, fine SAND (SP) - moist to wet

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
66	SS	19	480/610	11
71	SS	20	330/610	14
73	SS	21	510/610	22
76	SS	22	300/610	17
78	SS	23	410/610	17
81	SS	24	240/610	10
83	SS	25	510/610	2
86	SS	26	300/610	5
88	SS	27	250/610	25
91	SS	28	460/610	44
93	SS	29	530/610	68
96	SS	30	510/610	75

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

BH 7

CLIENT: S. Larkin Developments Inc.
PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON
DATUM:
DATES: BORING: June 8 to June 10, 2016
WATER LEVEL: August 30, 2016
TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
30	248.2	Compact to dense, grey, fine SAND (SP) - moist to wet			99	SS	31	~~330~~ / 610	22	
					100–101	SS	32	560 / 610	48	
31					102–103	SS	33	610 / 610	30	
31.9	246.3	END OF BOREHOLE at 31.9 m below existing grade. Groundwater level was measured in monitoring well, MW7-S, at 0.76 m below existing grade on August 30, 2016.			104					

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

Depth (m): 30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40

Depth (ft): 99, 100, 101, 102, 103, 104, 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 121, 122, 123, 124, 125, 126, 127, 128, 129, 130, 131

□ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

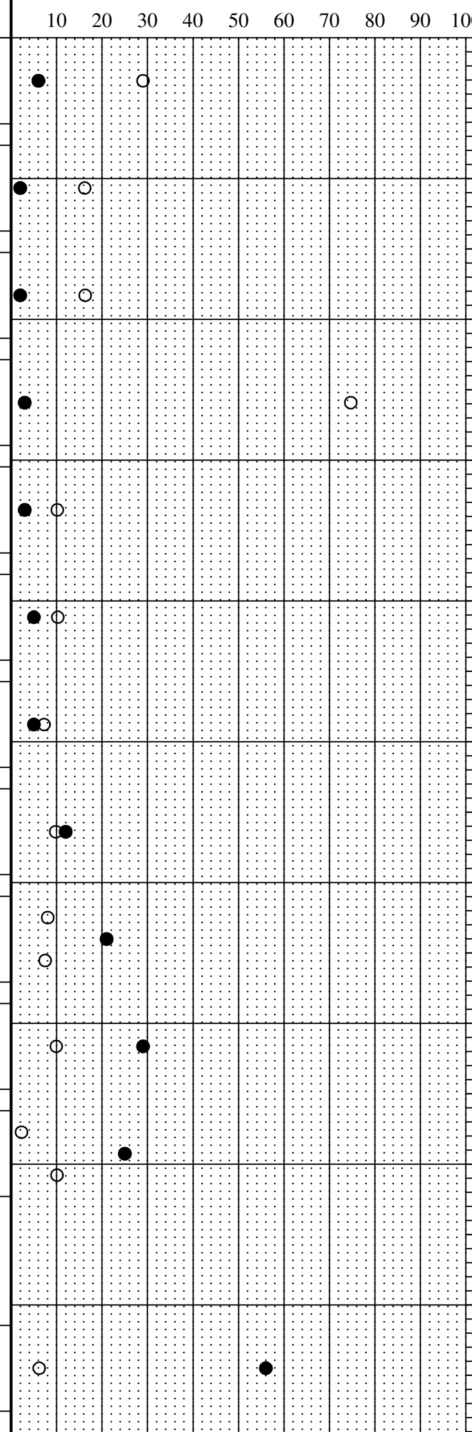
△ Pocket Penetrometer Test, kPa

CLIENT: S. Larkin Developments Inc. PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON DATUM:
DATES: BORING: June 6 and 7, 2016 WATER LEVEL: August 30, 2016 TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	**277.7**	
	277.5	230 mm TOPSOIL
		FILL: brown and grey silty sand and sandy silt - moist to wet - with occasional organic inclusions
	275.4	FILL: dark brown, topsoil - wet
	274.5	FILL: grey, sandy silt - moist - occasional sand and gravel layers with asphalt debris
	272.5	Compact, grey, sandy SILT (ML), TILL - trace gravel - some clay - moist
	270.0	Very stiff, grey, silty CLAY (CL-ML), some sand, trace gravel, TILL - moist
10		Continued Next Page

TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
SS	1	460/610	6	
SS	2	410/610	2	8 46 33 13
SS	3	360/610	2	
SS	4	320/610	3	
SS	5	470/610	3	
SS	6	610/610	5	6 37 43 14
SS	7	600/610	5	
SS	8	560/610	12	Non Plastic 3 34 45 18
SS	9	420/610	21	
SS	10	480/610	29	1 26 58 15
SS	11	480/610	25	0 11 66 23
SS	12	500/610	56	

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200
WATER CONTENT & ATTERBERG LIMITS (W_P, W, W_L)
DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼
STANDARD PENETRATION TEST, BLOWS/0.3m ●



□ Field Vane Test, kPa
■ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

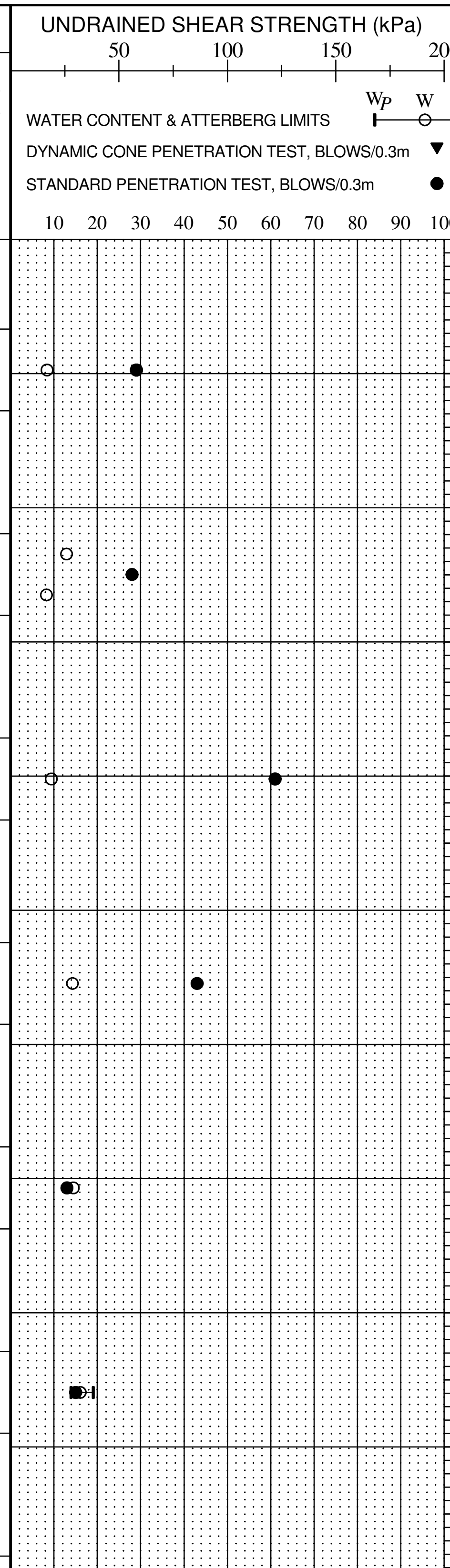
BH10

CLIENT S. Larkin Developments Inc. PROJECT No. 160622415

LOCATION 5433 Old Brock Road, Pickering, ON DATUM

DATES: BORING June 6 and 7, 2016 WATER LEVEL August 30, 2016 TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10	267.7	Very stiff to hard, grey silty CLAY (CL-ML), some sand ,trace gravel, TILL - moist	33					
11			36	SS	13	610/610	29	1 12 68 19
12	265.5	Compact, grey, silty SAND (SM), trace clay, trace gravel, TILL - moist to wet	41	SS	14	610/610	28	2 71 21 6
13	264.0	Very dense, grey SILT (ML) - some sand and clay - moist	46	SS	15	530/610	61	Non Plastic 0 11 73 16
14	262.9	Hard to stiff, grey silty CLAY (CL-ML) - trace sand - moist to wet	51	SS	16	550/610	43	
17			56	SS	17	360/610	13	
18			61	SS	18	530/610	15	0 5 67 28
20			65					



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□ Field Vane Test, kPa
■ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

Stantec

BOREHOLE RECORD

BH10

CLIENT: S. Larkin Developments Inc.

PROJECT No.: 160622415

LOCATION: 5433 Old Brock Road, Pickering, ON

DATUM:

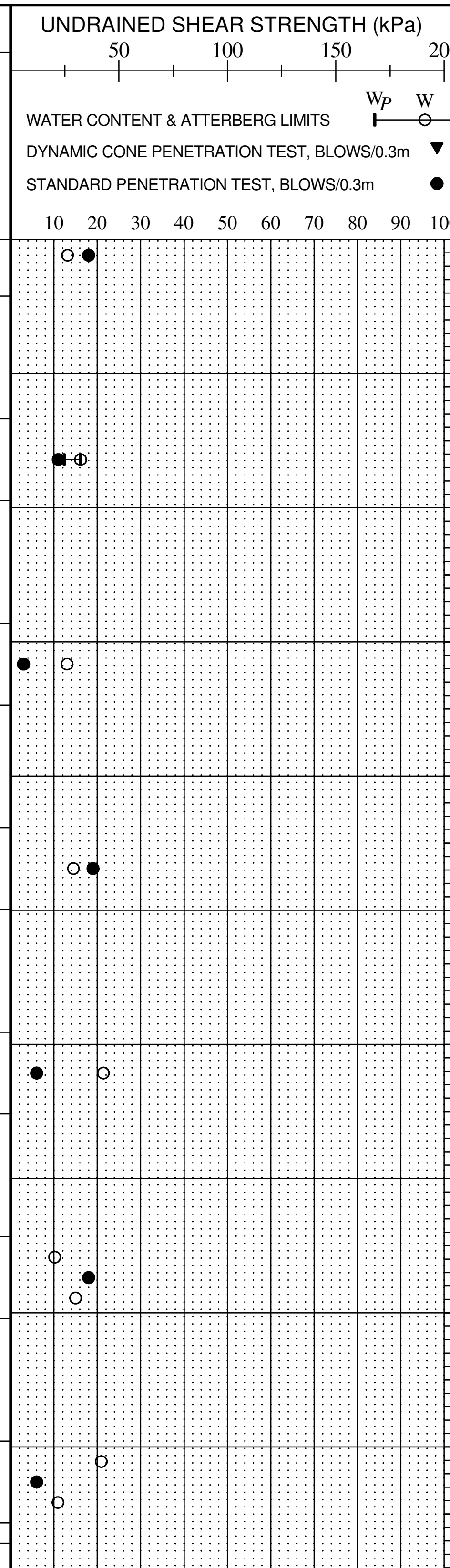
DATES: BORING: June 6 and 7, 2016

WATER LEVEL: August 30, 2016

TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
20	257.7	Very stiff to stiff, grey silty CLAY (CL-ML), with sand - moist to wet - soft at 22.8 m
24.4	253.3	Compact, grey silty SAND (ML) - trace clay - occasional fine sand layers - wet
25.9	251.8	Loose to compact, grey, SILT with sand (ML), some clay - moist to wet

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66	SS	19	150/610	18	
71	SS	20	560/610	11	0 21 60 19
76	SS	21	150/610	3	
81	SS	22	300/610	19	0 74 22 4
86	SS	23	610/610	6	
91	SS	24	530/610	18	Non Plastic 0 26 56 18
96	SS	25	610/610	6	



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□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

Stantec

BH10

CLIENT S. Larkin Developments Inc.

PROJECT No. 160622415

LOCATION 5433 Old Brock Road, Pickering, ON

DATUM

DATES: BORING June 6 and 7, 2016 WATER LEVEL August 30, 2016

TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
30	**247.7**									
	247.4				99	SS	26	~~330~~ 610	8	
		END OF BOREHOLE at approximately 30.3 m below existing grade. Groundwater level was measured in monitoring well, MW10-D, at 2.1 m below existing grade on August 30, 2016.								

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

□ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

TEST PIT RECORD

TP 1

CLIENT: S. Larkin Developments Inc.
PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON
DATUM:
EXCAVATION DATE: June 10, 2016
WATER LEVEL:
TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	263.9	700 mm, TOPSOIL, dark brown
0.7	263.2	Brown, silty CLAY (CL), sandy, trace gravel, TILL - moist
2.1	261.8	END of TEST PIT at approximately 2.1 m below existing grade. Test Pit dry upon completion of excavation.

SAMPLES

TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
GS	1			
GS	2			
GS	3			3 37 41 19
GS	4			
GS	5			8 31 43 18

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

Stantec

TEST PIT RECORD

TP 2

CLIENT: S. Larkin Developments Inc.

PROJECT No.: 160622415

LOCATION: 5433 Old Brock Road, Pickering, ON

DATUM:

EXCAVATION DATE: June 10, 2016

WATER LEVEL:

TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0	**281.4**	30 mm, TOPSOIL			0					
		FILL: brown, silty clay, - wet			1	GS	1			
		- with occasional topsoil and organic inclusions			2					
1					3					
					4	GS	2			0 18 64 18
					5					
					6	GS	3			
2	279.3	END of TEST PIT at approximately 2.1 m below existing grade.			7					
		Test Pit dry upon completion of excavation.			8					
3					9, 10, 11					
4					12, 13, 14					
5					15, 16					

UNDRAINED SHEAR STRENGTH (kPa): 50 100 150 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

Sample	Water content (%)
GS 1	30
GS 2	21
GS 3	12

□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

TEST PIT RECORD

Stantec

TP 3

CLIENT: S. Larkin Developments Inc.
PROJECT No.: 160622415
LOCATION: 5433 Old Brock Road, Pickering, ON
DATUM:
EXCAVATION DATE: June 10, 2016
WATER LEVEL:
TPC ELEVATION:

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	WATER CONTENT (%)
0	276.3	300 mm, TOPSOIL			0	GS	1			~17
	276.0	FILL: silty clay - wet			1	GS	2			~7
					3	GS	3			~14
					5	GS	4			~17
	274.5	Dark brown TOPSOIL		▽ (2.3 m)						
	273.9	END of TEST PIT at approximately 2.4 m below existing grade. Groundwater encountered at 2.3 m below existing grade upon completion of excavation.								

UNDRAINED SHEAR STRENGTH (kPa): 50, 100, 150, 200

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

DYNAMIC CONE PENETRATION TEST, BLOWS/0.3m ▼

STANDARD PENETRATION TEST, BLOWS/0.3m ●

10 20 30 40 50 60 70 80 90 100

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

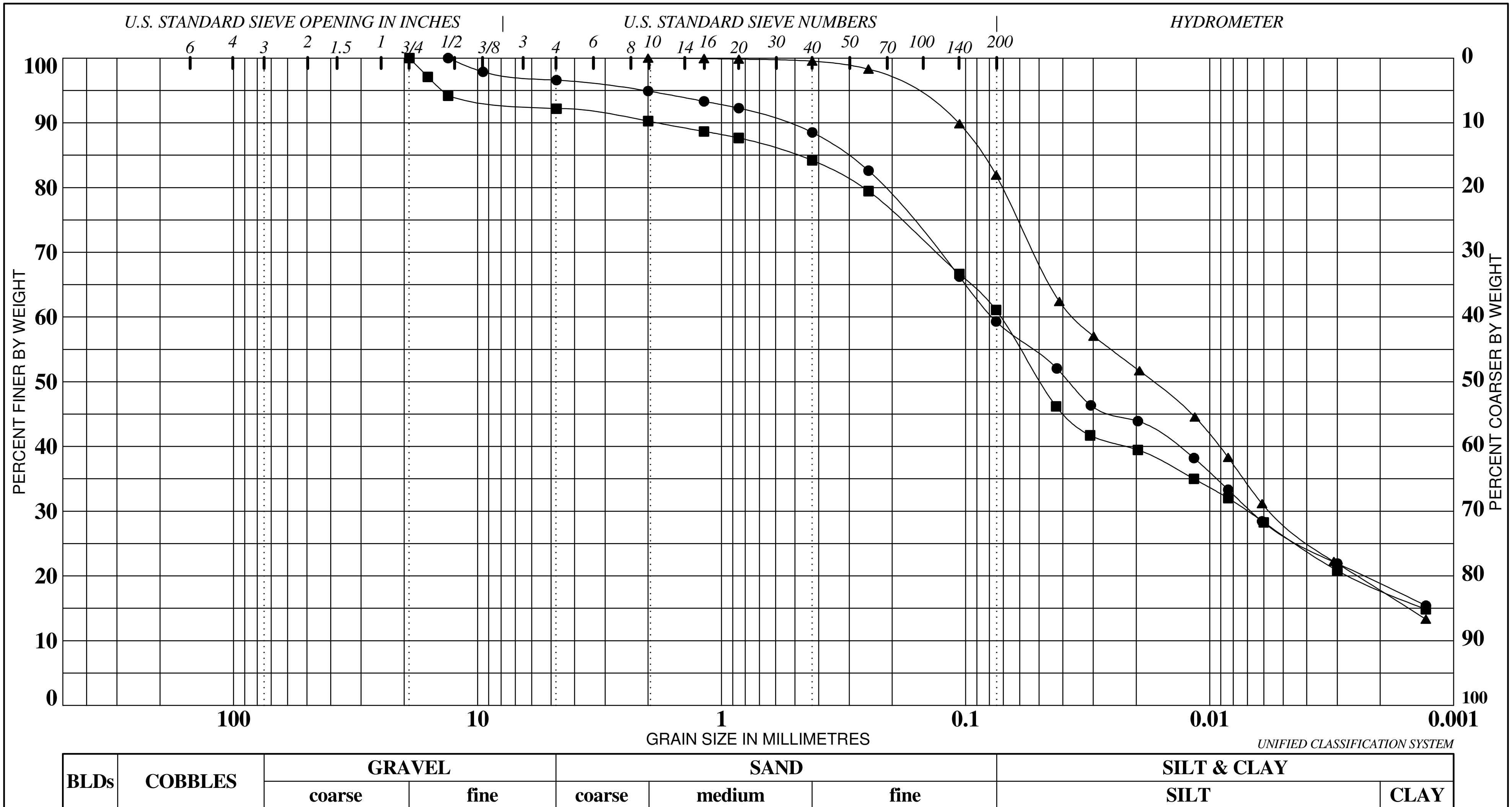
□ Field Vane Test, kPa

■ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

APPENDIX D

Geotechnical Laboratory Test Results



Sample	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
● TP 1	0.8	SILTY CLAY (CL), SANDY , trace gravel, TILL	11				3	37	41	19
■ TP 1	1.8	SILTY CLAY (CL), SANDY, trace gravel, TILL	12				8	31	43	18
▲ TP 2	1.1	FILL: silty clay	21				0	18	64	18

Stantec

Project: Claremont North Business Park

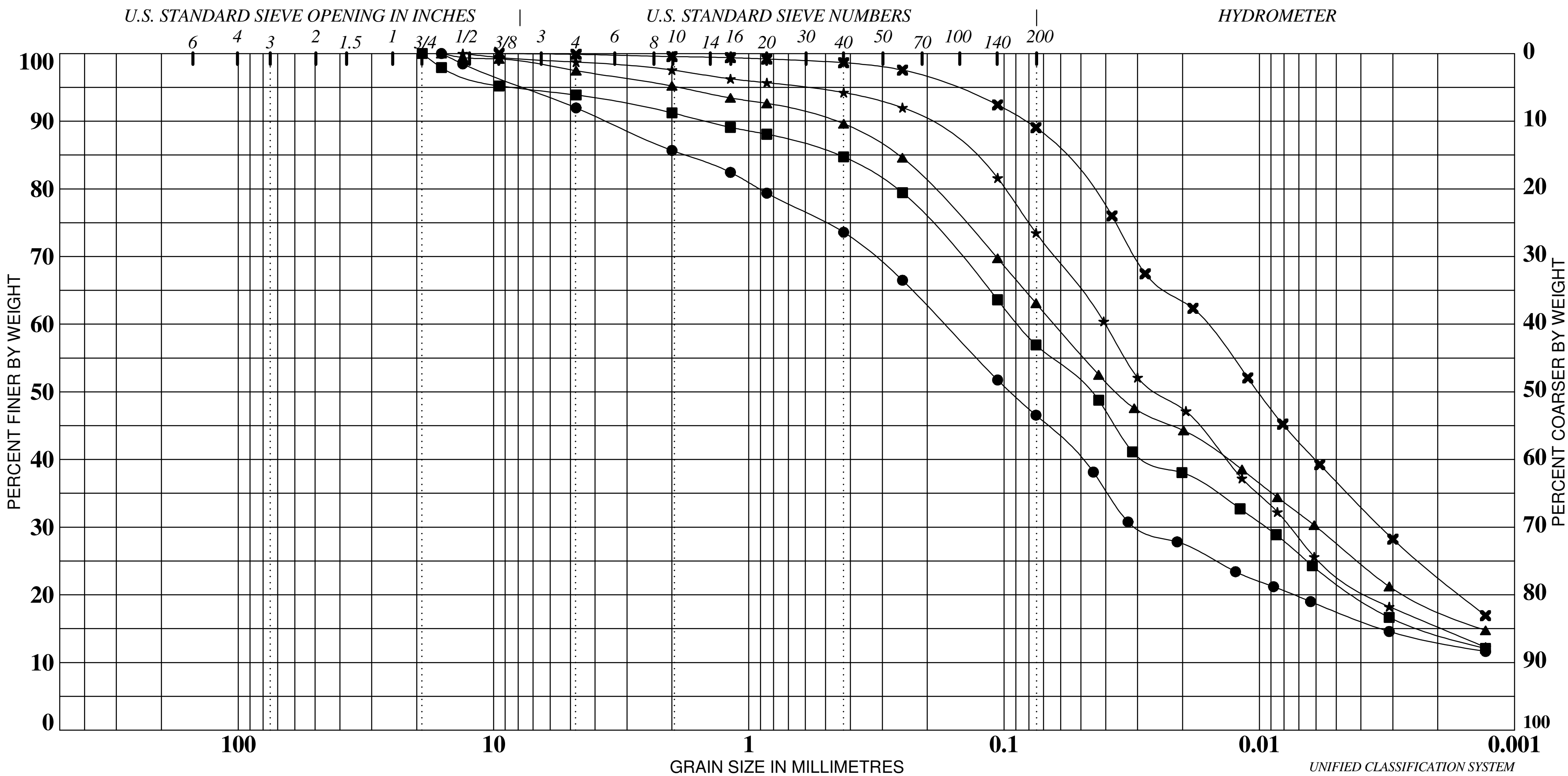
Location: 5433 Old Brock Road, Pickering, ON

Project No.: 160622415

GRADATION CURVE (ASTM D422)

Figure: 1

Remarks:



COBBLES	GRAVEL		SAND			SILT and CLAY	
	coarse	fine	coarse	medium	fine	SILT	CLAY

	Specimen	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
●	BH10	1.1	FILL: silty sand	16				8	45	33	13
■	BH10	4.1	FILL: silty sand	10				6	37	43	14
▲	BH10	5.6	SANDY SILT (ML), some clay, trace gravel, TILL	10				3	34	45	18
★	BH10	7.2	SANDY SILT (ML), some clay, trace gravel, TILL	10				1	25	58	15
✖	BH10	8.1	SILTY CLAY (CL), some sand, TILL	10				0	11	66	23

Stantec

Project: Claremont North Business Park

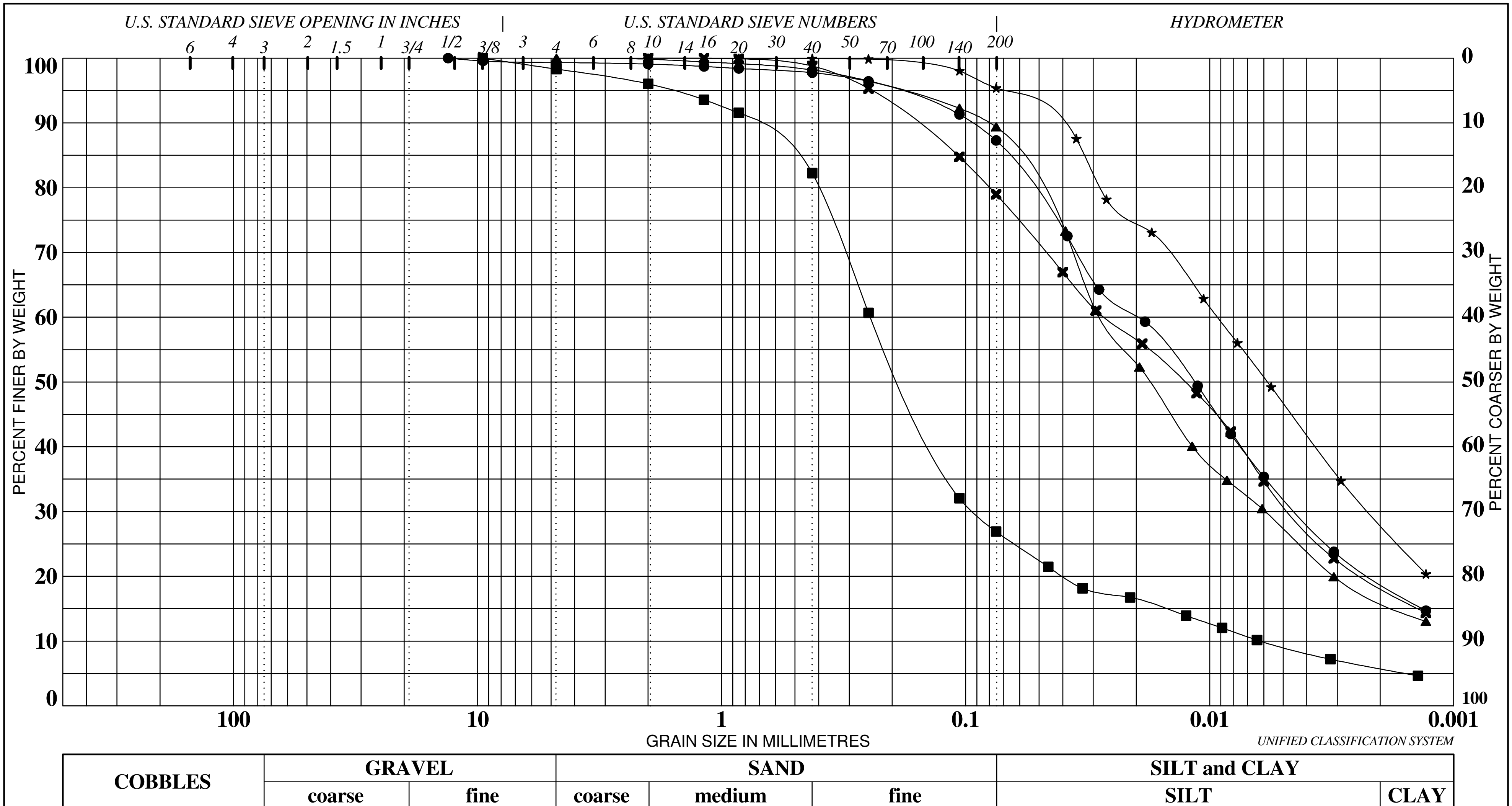
Location: 5433 Old Brock Road, Pickering, ON

Project No.: 160622415

GRADATION CURVE (ASTM D422)

Figure: 1

Remarks:



Specimen	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
● BH10	11.0	SILTY CLAY (CL), some sand, trace gravel, TILL	8				1	12	68	19
■ BH10	12.3	SILTY SAND (SM), trace clay and gravel, TILL	13				2	71	21	6
▲ BH10	14.0	SILT (ML), some sand and clay	9	NP	NP	NP	0	11	73	16
★ BH10	18.6	SILTY CLAY (CL-ML), trace sand	16	19	14	5	0	5	67	28
✖ BH10	21.6	SILTY CLAY (CL-ML), some sand	16	16	12	4	0	21	60	19

Stantec

Project: Claremont North Business Park

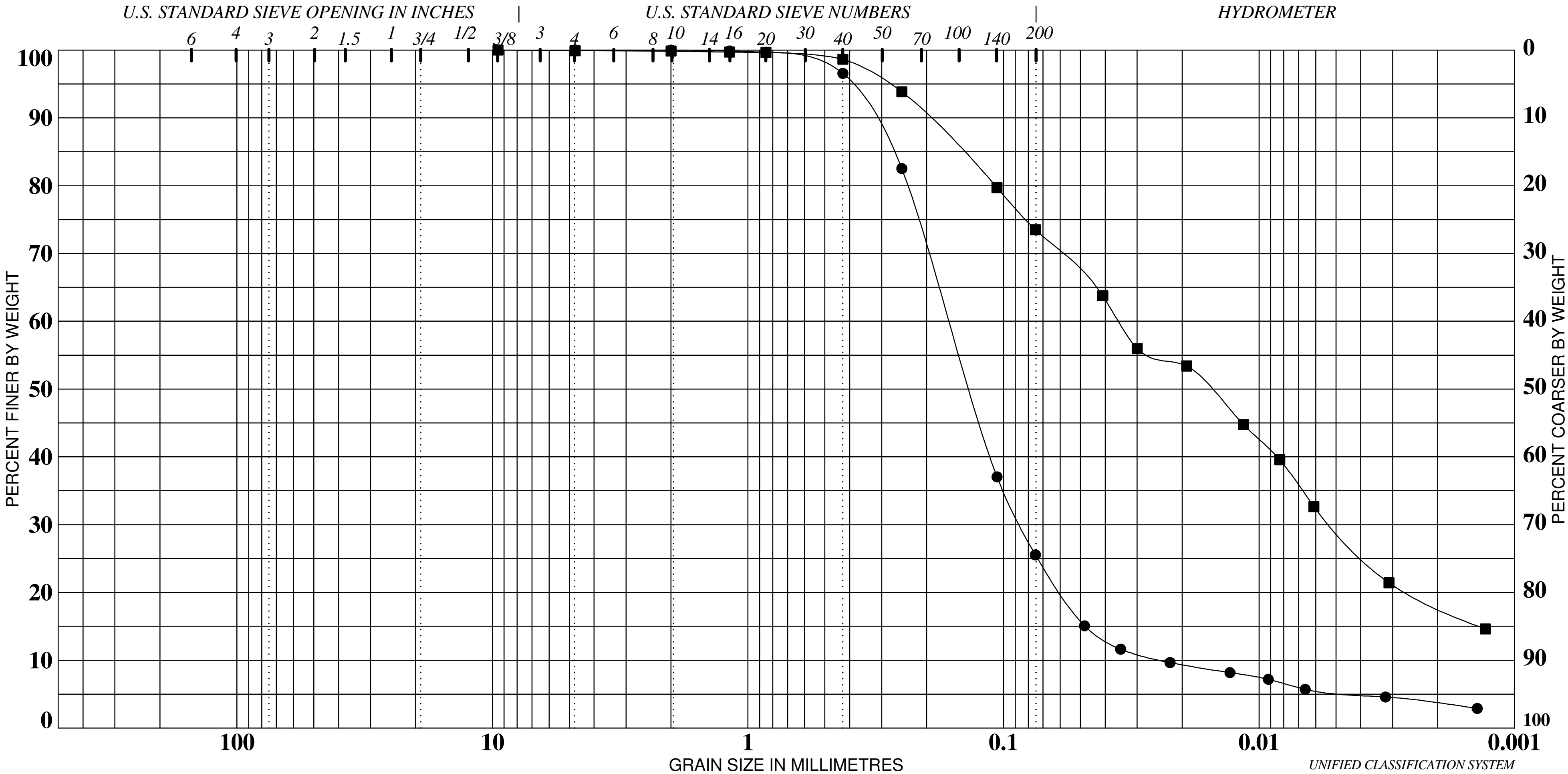
Location: 5433 Old Brock Road, Pickering, ON

Project No.: 160622415

GRADATION CURVE (ASTM D422)

Figure: 2

Remarks:



COBBLES	GRAVEL		SAND			SILT and CLAY	
	coarse	fine	coarse	medium	fine	SILT	CLAY

	Specimen	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
●	BH10	24.7	SILTY SAND (ML), trace clay	14				0	74	22	4
■	BH10	27.9	SILT with SAND (ML), some clay	15				0	26	56	18

Stantec

Project: Claremont North Business Park

Location: 5433 Old Brock Road, Pickering, ON

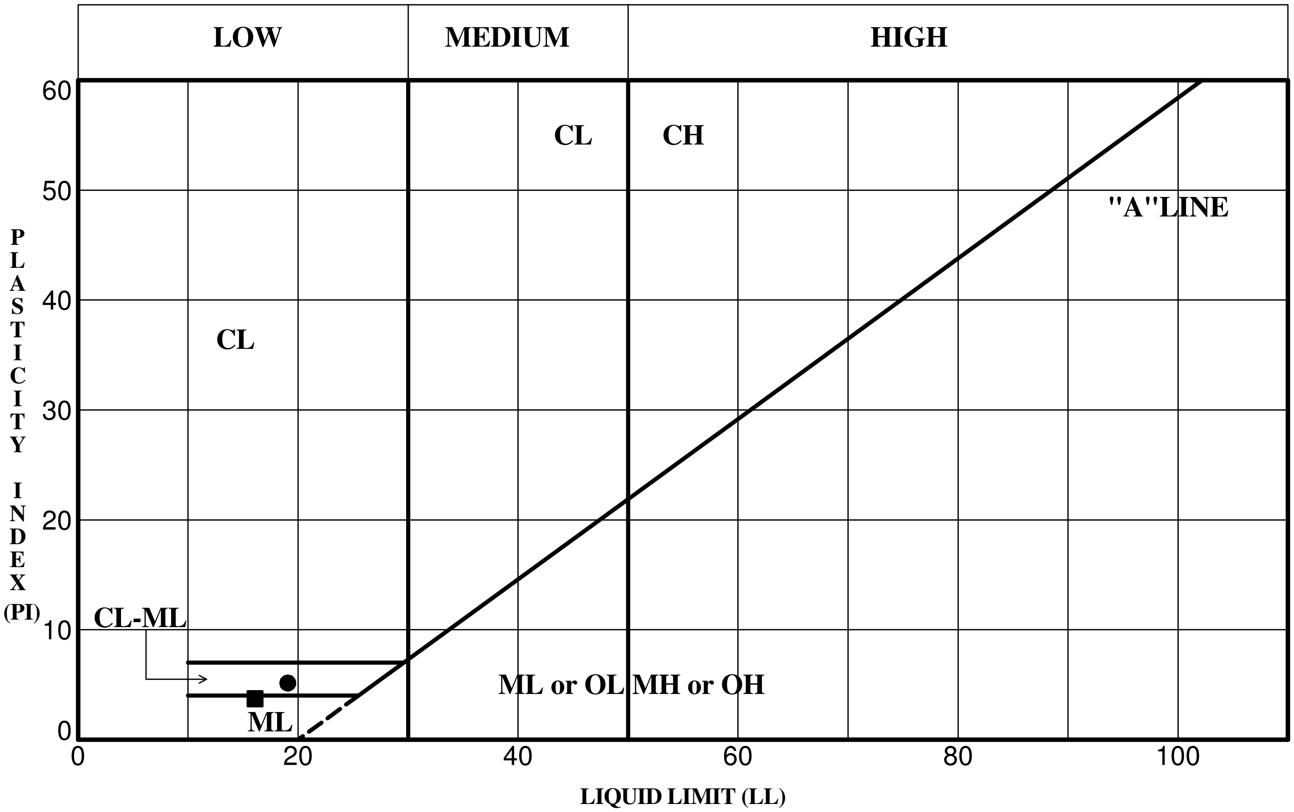
Project No.: 160622415

GRADATION CURVE (ASTM D422)

Figure: 3

Remarks:

PLASTICITY CHART



	Specimen	Depth (m)	LL	PL	PI	Fines	W%	Classification
●	BH10	18.6	19	14	5	95	16	SILTY CLAY (CL-ML), trace sand
■	BH10	21.6	16	12	4	79	16	SILTY CLAY (CL-ML), some sand

STN13-ATTERBERG 160622415.GPJ MM.GDT 11/4/16

Stantec

Project: Claremont North Business Park

Location: 5433 Old Brock Road, Pickering, ON

Project No.: 160622415

ATTERBERG LIMITS
(ASTM D4318)

Figure: 4

Remarks: