Geotechnical Investigation Report - 1854 Liverpool Road, Pickering, Ontario

2019-07-22

Prepared for:
Grant Morris Associates Ltd. on behalf of Mr. Alireza Adjedani

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1.0 INTRODUCTION

Cambium Inc. (Cambium) was retained by Grant Morris Associates Ltd. on behalf of Mr. Alireza Adjedani (Client) to complete a geotechnical investigation in support of the design and construction of 16-storey mixed use building located at 1854 and 1858 Liverpool Road in Pickering, Ontario (Site).

The Site is located approximately 200 m north of intersection of Kingston Road and Liverpool Road in Pickering, Ontario. The purpose of this geotechnical investigation was to obtain information about the subsurface conditions by means of a number of boreholes and based on the findings provide recommendations pertaining to the geotechnical design of the proposed high-rise building. This report presents the methodology and findings of the geotechnical investigation at the Site and addresses requirements and constraints for the design and construction of the proposed structure and facilities.

2.0 METHODOLOGY

2.1 Borehole Investigation

Cambium completed a geotechnical investigation at the Site on June 7, 2019. A total of Three (3) boreholes, designated as BH (MW)101-19 through BH (MW)103-19, were advanced into the subsurface at predetermined locations throughout the Site. BH101-19 and BH102-19 were terminated at depths of 14.2 m below ground surface (bmgs). BH103-19 was terminated at depth of 12.6 mbgs. One (1) additional borehole (BH104-19) was advanced on July 3 to confirm the bedrock surface elevation and core the bedrock. The location of the boreholes was obtained from a handheld GPS unit while the elevation of the boreholes was surveyed relative to BM (porch at the front door of the existing house). The elevation of the top of porch at the northeast corner has a geodetic elevation of 89.671 masl according to the survey plan provided by the client. A Site Plan, including borehole locations and benchmark is appended as Figure 1 of this report.

Drilling and sampling was completed using a track-mounted drill rig operating under the supervision of a Cambium technician. The boreholes were advanced to the sampling depths by means of continuous flight solid or hollow stem augers with 50 mm O.D. split spoon samplers. Standard Penetration Test (SPT) N values were recorded for the sampled intervals as the number of blows required to drive a split spoon sampler 305 mm into the soil, using a 63.5 kg drop hammer falling 750 mm, as per ASTM D1586 procedures. The SPT N values are used in this report to assess consistency of cohesive soils and relative density of non-cohesive materials. Soil samples were collected at approximately 0.75 m intervals in the upper 3.0 m depth and in 1.5 m intervals below 3.0 m depth. Bedrock coring involved one 1.5 m long rock core obtained from borehole BH104-19 using a 63.5 mm diameter HQ diamond core rock sampler to assess the quality of the bedrock onsite. The encountered soil units were logged in the field using visual and tactile methods, and samples were placed in labelled plastic bags for transport, future reference, possible laboratory testing, and storage.

Open boreholes were checked for groundwater and general stability prior to backfilling. Borehole BH101-19, BH102-19 and BH103-19 were outfitted as monitoring wells in order to understand the groundwater conditions at the site. All other boreholes were backfilled and sealed in accordance with Ontario Regulation (O.Reg.) 903, as amended, and the property was reinstated to pre-existing conditions.

Borehole logs are provided in Appendix A. Site soil and groundwater conditions are described and geotechnical recommendations are discussed in the following sections of this report.

2.2 Physical Laboratory Testing

Physical laboratory testing, including Five (5) particle size distribution analyses (LS-702,705), and Two (2) tests of Atterberg limits (LS-703,704), was completed on selected soil samples to confirm textural classification and to assess geotechnical parameters. Moisture content testing was completed on all soil samples. Testing results are presented in Appendix B and are discussed in Section 3.0.

3.0 SUBSURFACE CONDITIONS

The detailed soil profiles encountered in the boreholes are indicated on the attached borehole logs in Appendix A. It should be noted that the conditions indicated on the borehole logs are for specific locations only, and can vary between and beyond the borehole locations. The soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change. In addition, the descriptions provided in the borehole logs are inferred from a variety of factors, including: visual observations of the soil samples retrieved, laboratory testing, measurements prior to and after drilling, and the drilling process itself (drilling speed, shaking/grinding of the augers, etc.).

Based on the results of the borehole investigation, subsurface conditions at the Site generally consist of topsoil overlying deposits of primarily silt rich soils, with some layers of very loose sand and silt materials overlaying silty sand till. Bedrock was encountered and cored at BH104-19 during the additional geotechnical investigation.

3.1 Topsoil

A topsoil layer, ranging in thickness from 150 mm to 330 mm, was encountered at all of the borehole locations excluding BH104-19. The topsoil, which can be generally described as sand, with some silt, contained organics. Topsoil was observed to be brown in colour and moist at the time of the investigation. Analysis of the organic or nutrient content of the topsoil was not part of the scope of this investigation.

3.2 Silt and Clay

Beneath the topsoil, a layer of silt and clay was found in all boreholes, the thickness of the silt and clay is between 5.0 m and 6.0 m. Standard Penetration Test (SPT) “N”-values measured within this zone ranged from 4 to 17 blows per 0.3 m of penetration, and as such, the consistency of the silt and clay crust ranges from firm to stiff. The natural water content of the silt and clay within this zone ranges from 10.6% to 25.8%.

Laboratory particle size distribution analyses were completed for two (2) samples of the silt and clay material, taken from depths of between 0.8 mbgs and 2.0 mbgs The analysis results are summarized in Table 1 and provided in Appendix B.

Table 1 Particle Size Distribution Analysis – Silt and Clay

Borehole	Depth (mbgs)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH103-19-SS2	0.8 – 1.2	Silt and Clay, trace Sand	0	1	61	38	23.2
BH101-19-SS3	1.5 – 2.0	Silt and Clay, trace Sand	0	1	56	43	23.2

Atterberg Limits tests were performed on these two (2) representative samples of soils in addition to the grain size analysis tests. The results of the Atterberg Limits tests are presented in Appendix B as well as Table 2 below.

Table 2 Atterberg Limits Analysis Test Results Summary

Borehole	Depth (mbgs)	Soil	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Classification
BH103-19-SS2	0.8 – 1.2	Silt and Clay, trace Sand	30.3	16.7	13.6	CL
BH101-19-SS3	1.5 – 2.0	Silt and Clay, trace Sand	33.0	16.2	16.7	CL

3.3 Gravelly Sand

Gravelly sand was encountered in BH103 at a depth of 3.0 mbgs to 5.5 mbgs. This soil was presented in a damp to moist in-situ condition with moisture contents varying from 9.7% to 14.0% and with a loose to compact relative density based on SPT N values of between 11 and 28 per 0.3 m of penetration.

A laboratory particle size distribution analysis was completed for one (1) sample of the gravelly sand material, taken from a depth of between 4.6 mbgs and 5.0 mbgs. The analysis results are summarized in Table 3 and provided in Appendix B.

Table 3 Particle Size Distribution Analysis – Gravelly Sand

Borehole	Depth (mbgs)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH103-19-SS6	4.6 – 5.0	Silty Sand, trace clay, some gravel	26	50	20	4	9.7

3.4 Sand and Silt

Beneath the silt and clay (BH101 and BH102), a layer of sand and silt was found and extended to depths of between 9.0 m and about 11.0 mbgs. Standard Penetration Test (SPT) “N”-values measured within this zone ranged from 1 to 4 blows per 0.3 m of penetration, indicating a compactness from very loose to loose. The natural water content of the sand and silt within this zone ranges from 8.0% to 12.0%.

A laboratory particle size distribution analysis was completed for one (1) sample of the sand and silt material, taken from depths of between 9.1 mbgs to 9.6 mbgs. The analysis results, based on the Unified Soil Classification System (USCS) scale, are summarized below in Table 4, with full results provided in Appendix B.

Table 4 Particle Size Distribution Analysis – Sand and Silt

Borehole	Depth (mbgs)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH101-19-SS9	9.1 – 9.6	Sand and Silt, trace clay, trace gravel	4	54	37	5	12.0

3.5 Silty Sand Till

A silty sand till with various amount of clay and gravel was encountered in all boreholes from 11.0 mbgs to the termination depth of 12.0 mbgs to 14.0 mbgs (BH101 to BH103). The till was found imbedded with shale complex below 16.5 mbgs in BH104. The till was damp to moist at the time of the investigation, with a natural moisture content between 8.4% and 11.7%. SPT N values obtained were from 20 to more than 50 blows per 0.30 m of penetration, indicating a compact to very dense soil matrix.

A laboratory particle size distribution analysis was completed for one (1) sample of the silty sand till material, taken from depths of between 13.7 mbgs to 14.2 mbgs. The analysis results, based on the Unified Soil Classification System (USCS) scale, are summarized below in Table 5, with full results provided in Appendix B.

Table 5 Particle Size Distribution Analysis – Silty Sand

Borehole	Depth (mbgs)	Soil	% Gravel	% Sand	% Silt	% Clay	% Moisture Content
BH103-19-SS10	13.7 – 14.2	Silty Sand, some clay, some gravel	16	41	33	10	8.0

3.6 Bedrock

Borehole BH104-19 was extended to bedrock refusal at a depth of 22.3 mbgs and was cored 1.5 m into the bedrock. The upper 0.3 m of core recovered has a grey, soft, crumbly appearance, with some soil obtained from within fractures and fissures in the rock. This portion of the core retained no pieces greater than 10 cm in length, resulting in an RQD of 0%, indicating extremely poor-quality rock. The lower 1.2 m of core recovered, contained a medium to dark grey limestone with a weathered surface and increasing competency with depth. The limestone rock core taken from BH104-19 had an RQD of 57% with 91% total core recovery, providing evidence of fair quality rock.

3.7 Groundwater

All boreholes were found to have some groundwater seepage upon completion. The water elevations were measured on completion of drilling and prior to backfill. During drilling, short-term groundwater table was found at depths of 3.3 m to 6.1 m below ground surface as listed on Table 6 below. The stabilized groundwater levels observed in the monitoring wells on June 13, 2019 were at depths ranging from 2.3 mbgs to 3.3 mbgs, corresponding to Elevations from 86.6 masl to 87.6 masl, as listed on Table 6. It should be noted that the groundwater levels at the site may fluctuate seasonally and in response to climatic events.

Table 6 Groundwater Levels Observed in Boreholes

Borehole	Date of Drilling	Date of Observation	Depth of Groundwater (mbgs)	Notes
MW101-19	June 06, 2019	June 06, 2019 June 13, 2019	6.1 3.3	During drilling Monitoring Well
MW102-19	June 06, 2019	June 06, 2019 June 13, 2019	3.3 3.0	During drilling Monitoring Well
MW103-19	June 07, 2019	June 07, 2019 June 13, 2019	3.6 2.3	During drilling Monitoring Well
BH104-19	July 03, 2019	July 03, 2019	3.6	During drilling

4.0 GEOTECHNICAL CONSIDERATIONS

The following recommendations are based on the borehole information and are intended to assist designers. Recommendations should not be construed as providing instructions to contractors, who should form their own opinions about site conditions. It is possible that subsurface conditions beyond the borehole locations may vary from those observed. If significant variations are found before or during construction, Cambium should be contacted so that we can reassess our findings, if necessary.

4.1 General Site Preparation

All topsoil, organics and deleterious material should be removed from below the building areas prior to construction. For site grading, in areas of cut or minor fill where the proof roll and/ or inspection has identified unsuitable subgrade conditions, whether too soft or too wet, material is to be removed and replaced with an approved OPSS 1010 Granular ‘B’ Type I compacted material, under guidance of Cambium Staff.

4.2 Excavations

Temporary excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). As per the OHSA, excavations less than 1.2 m deep can have unsupported vertical walls. The soils at this site would generally be classified as Type 3 soils in accordance with OHSA, with unsupported side slopes no steeper than 1H:1V to the bottom of the excavation. For the soils below the water table, and the very soft to soft and very loose to loose soils are classified as Type 4 soils. Where excavations are carried out in Type 4 soils, the slopes should not exceed 3H:1V, therefore shoring will likely be required.

Excavation side slopes should be protected from exposure to precipitation and associated ground surface runoff and should be inspected regularly for signs of instability. If localized instability is noted during excavations or if wet conditions are encountered, the side slopes should be flattened as required to maintain safe working conditions or excavation sidewalls must be fully supported (shored). It is expected however, the excavation will extend to about

7.0 mbgs to 8.0 mbgs for two-levels of underground parking construction, and given the close proximity of the adjacent roads and properties/buildings, that it will be necessary to shore this excavation. Guidelines on excavation shoring are provided in Section 4.2.1 of this report.

4.2.1 Excavation Shoring

A shoring system should be designed to protect adjacent structures and services. As discussed in previous section, it is expected that the excavation will extend to about 7.0 mbgs to 8.0 mbgs.

In general, there are three basic shoring methods that are commonly used in local construction practice:

- Steel soldier piles with timber lagging;
- Driven steel sheet piles; and,
- Continuous concrete secant pile (caisson) walls.

The geotechnical parameters, which are considered to be applicable for the shoring design, are as follows:

Table 7 Soil Parameters for Earth Pressure

Soil	Bulk Unit Weight γ (kN/m^3)	Internal Friction Angle Φ' (°)	Active earth pressure coefficient Ka (Rankine)	Passive earth pressure coefficient Kp (Rankine)	At-rest earth pressure coefficient Ko (Rankine)
Firm to stiff silt and clay (0.0 m– 7.0 m)	19.5	28	0.36	2.77	0.53
Very loose to loose sand and silt (7.0 m – 11.5 m)	20.0	24	0.42	2.38	0.59
Dense to very dense silty sand till (11.5 m – 15.6 m)	20.5	32	0.31	3.26	0.47

For all types of shoring systems, some form of lateral support to the wall is required for excavation of this site. Lateral restraint could be provided by means of tie-backs consisting of soil anchors or grouted bedrock anchors. It should be noted that the use of soil or rock anchor tie-backs would require the permission of the adjacent building or roadway owners since the anchors would be installed beneath their properties.

Soil anchors may be designed by using average bond value of 40 kPa. It is re-iterated that subsurface conditions may vary beyond the site's confines. As a result, the design values must be confirmed by load tests, carried out to twice the design load. The final design value should be reviewed by Cambium.

Alternatively, interior struts may be considered, connected either to the opposite side of the excavation or to raker piles and/or footings within the excavation. However internal struts could interfere with the construction of the foundations and superstructure.

It should be noted that groundwater may be encountered during soldier pile/caisson construction, and the contractor must be prepared to deal with water seepage into the caisson shafts without undue delays. Due to the groundwater and wet silty soils or perched water in the shallow layer, it might be difficult to prevent groundwater from penetrating into the excavation through gaps in timber lagging.

It is imperative that an experienced shoring engineer must be retained to design the shoring system prior to commencement of construction. The final shoring design should be reviewed by Cambium. In addition, a pre-construction survey of the surrounding structures/roads is recommended prior to commencement of shoring construction. The shoring system and surrounding structures must be monitored for horizontal and vertical movements, prior to, during and after the excavation.

4.3 Dewatering

As discussed in the previous section, groundwater was observed in monitoring well at depths of between 2.3 mbgs and 3.3 mbgs. It should be noted that the groundwater table is influenced by seasonal fluctuations and major precipitation events.

It is understood the proposed development will involve deep excavations. Overall, it is anticipated that significant groundwater seepage is likely to occur where excavations are made below the groundwater level. For excavations within the silt and clay, it is expected that, due to the relatively low hydraulic conductivity of this material, it should be possible to handle the groundwater inflow from this deposit by pumping from well filtered sumps in the floor of the excavation, using suitably sized pumps. A dewatering system such as well-points may also be used to depress the groundwater level below the excavation bases. If a well-point dewatering system is used, registration on the Environmental Activity and Sector Registry (EASR) or a Permit to Take Water (PTTW) is may be required from the Ministry of the Environment Conservation and Parks (MOECP) as pumping could exceed 50,000 L/day or 400,000 L/day respectively.

4.4 Foundation Design

In general, the overburden conditions at this site consist of a deposit of silt and clay overlaying silty sand till deposit. The till material become dense to very dense with increased depth. The surface of the very dense ("100-blow") soils exists at approximately 15.6 mbgs (74.3 masl). The bedrock surface is inferred to be at depth of 22.3 mbgs (67.6 masl).

It is our understanding that the proposed developments at the subject property would consist of a 16-storey building with 2-levels of underground parking. Design recommendations with respect to the building foundations are provided below based on the obtained information. The existing clay and silt at shallow depths and underlying very loose sand and silt deposits are considered unsuitable for supporting conventional spread footing foundations. It is recommended that the proposed multi storey building be founded on driven H-Piles or drilled shaft caissons seated within the very dense till material.

4.4.1 Steel H-Pile Foundations

Steel H-piles driven into very dense material (having SPT "N" values greater than 100 blows per 0.3 m of penetration) are a feasible method of supporting the proposed structure. The piles should be driven at least 4.0 m into the "100-blow" material. The recommended factored axial resistance at ULS for the HP 310x 110 piles driven into "100-blow" material may be taken as 800 kN, and the geotechnical axial reaction at SLS (for 25 mm of settlement) may be taken as 700 kN. The structural resistance of the pile must be checked by the structural engineer.

For the installation of steel H-piles, the piles should be reinforced at the tip with driving shoes such as OPSD 3000-201 (HP310 Oslo Point). The criteria of pile driving termination will be dependent on the pile driving equipment and length of pile. Therefore, the criteria should be established at the time of construction when the piling equipment is known. The criteria must also be selected to ensure that the piles are not overdriven to avoid possible damage to the piles. Piling operations should be inspected on a full-time basis by geotechnical personnel.

For lateral soil-pile interaction analysis, the horizontal subgrade reaction (lateral spring parameters) and ultimate lateral resistance may be calculated from the following expression if the soil is primarily cohesionless at the site:

$$ks = nh(z/d)$$

$$Pult = 3\gamma' z\ KP$$

Where ks = Coefficient of horizontal subgrade reaction (kPa/m);

nh = Constant of horizontal subgrade reaction (kPa/m);

d = Pile width (m); and

z = Depth (m)

γ' = effective unit weight (kN/m^3)

Where the soil is primarily cohesive, the coefficient of horizontal subgrade reaction and ultimate lateral resistance can be estimated from:

$$ks = 67\ Su/d$$

$$Pult = 9\ Su$$

Where Su = Undrained shear strength (kPa)

Table 8 below summarizes the values of nh and Su for the various soil types encountered in the borehole investigation.

Table 8 Design Parameters for Lateral Resistance

Soil Strata	Bulk Unit Weight (kN/m^3)	Passive Earth Pressure Coefficient	n_h (kPa/m)	Undrained Shear Strength (kPa)
Very loose to loose sand and silt (7.0 m – 11.5 m)	19.0	2.38	3000	-
Very dense silty sand Till to Shale Complex (11.5 m – 22.3 m)	20.0	3.2	12000	-
Limestone bedrock (>22.3 m)	22.0	-	-	250

Group action for lateral loading should be considered when the pile spacing in the direction of loading is less than 8d. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading using a reduction factor, R, as follows in Table 9:

Table 9 Reduction Factor Due to Pile Spacing

Pile Spacing d = Pile Diameter or Width	Subgrade Reaction Reduction Factor, R
8d	1.00
4d	0.75
2d	0.40
d	0.25

4.4.2 Drilled Shaft Caissons

A foundation system comprising drilled shaft caissons founded within the very dense silty sand till (having SPT “N” values greater than 100 blows per 0.3 m of penetration) The following geotechnical resistance of caissons may be used, assuming a minimum 4.0 m long socket into the “100-blow” till deposit for a 0.762 m diameter caisson, or a 3.0 m long socket into the “100-blow” till deposit for a 0.915 m diameter caisson, a larger diameter caisson may be considered if higher resistance are required. Recommended geotechnical resistance for the caissons to carry the vertical loads have been provided in Table 10.

Table 10 Geotechnical Resistance of Caissons

Caisson Diameter (m)	Socket length (m)	Factored Geotechnical Resistance at ULS (kN)	Geotechnical Reaction at SLS (kN)
0.762	4.0	2000	1800
	5.0	2200	2000
0.915	3.0	2600	2200
	3.5	2800	2500

The required caisson diameters for the proposed structure were assumed. In general, greater capacity could be obtained by increasing the diameter/size of the caisson.

The value of a Factored Geotechnical Resistance at ULS was assessed assuming a Consequence Factor of 1.0 (typical) and a Resistance Factor of 0.4 (typical). The geotechnical axial reaction at SLS is considering maximum 25 mm of settlement.

The resistance to lateral loading developed by the soils in front of the caissons and the reductions due to group effects may be determined as similar Driven H Pile.

The following preliminary and general comments regarding caisson foundations are provided for reference: The caisson contractor should be advised to provide temporary or permanent caisson liners to support the soils during construction, so that caissons can be adequately formed, cleaned and inspected prior to pouring concrete, and special methods such as the use of drilling mud and placement of concrete by tremie methods may be required near the bottom of the caissons to keep the hole open and minimize disturbance to the caisson base. All caisson bases should be installed in accordance with OPSS 903 and inspected by qualified geotechnical personnel to verify the competency of the founding surface. A high slump concrete (150 mm) should be used if no vibration is done. The mix design proposed for use should be submitted to Cambium for review and approval well in advance of start of work. Sampling and testing of concrete compressive strength cylinders to the requirements of CAN/CSA A23.1 and A23.2 is recommended. At least one set of concrete cylinders should be taken for each day caissons are poured.

4.5 Seismic Site Classification

The structures should be designed to withstand forces caused by seismic activity in accordance with the Ontario Building Code (OBC). In order to determine a site classification, it was assumed that soils as encountered in the samples retrieved in the boreholes would remain continuous minimum depth of 30 m below the bottom of any foundations. In addition, average 'N60' values for soils were assumed for the site. Based on these assumptions, in combination with the known local geological conditions, the site class for the proposed building is "D" as per Table 4.1.8.4.A, Site Classification for Seismic Site Response, OBC 2012. These earthquake/seismic design parameters should be reviewed in detail by the structural engineer and incorporated into the design as required.

4.6 Frost Penetration

Based on the Ontario Provincial Standard Drawing (OPSD) 3090.101, the typical frost penetration depth for the proposed structure is expected to be approximately 1.4 mbgs. Footings for the proposed structure should be situated at or below this depth for frost penetration or should be protected. If construction is carried out during the winter months, all footing locations where excavated soils become exposed to potential frost penetration should be poured within the day to prevent potential future settlements. If the footings cannot be poured within the same day the soils are excavated, the surface should be covered with thermal insulation equivalent to 1.4 m of soil cover to prevent potential freezing of the frost susceptible soils at the Site.

It is assumed that any pavement structure thickness will be less than 1.4 m; therefore, grading and drainage are important for good pavement performance and life expectancy. Any utilities should be located below this depth or be appropriately insulated.

4.7 Basement Floor Slabs

It is considered that conventional construction may be used for this building basement floor slab. Any soft, loose, wet, and disturbed material should be removed and replaced with quality granular material compacted to 100% of its SPMDD.

The guidelines below are based on the assumptions that a "drained" foundation system will be provided. These guidelines should be revisited if it is decided that the construction of the basement would be water-tight.

To prevent hydrostatic pressure building up beneath the floor and potential groundwater infiltration, it is suggested that the granular base for the floor be drained. Provision should be made for at least 300 mm of 600 mm clear crushed stone to underlie the floor. A vapour barrier beneath the slab should be provided. Perimeter and under floor drainage systems would be also required for the proposed building. The perforated pipes should discharge to a positive outlet such as a storm water sewer or a sump from which the water is pumped.

4.8 Backfill and Compaction

Excavated non-organic fill and native sand and silt soils from the site may be appropriate for use as fill below grading and parking areas, provided that the actual or adjusted moisture content at the time of construction is within a range that permits compaction to required densities. Some moisture content adjustments may be required depending on seasonal conditions. Geotechnical inspections and testing of engineered fill are required to confirm acceptable quality.

Engineered fill for foundations should consist of free-draining granular material meeting the specifications of OPSS 1010 Granular B or an approved equivalent, and should be placed in maximum 200 mm thick lifts compacted to a minimum of 100 percent standard Proctor maximum dry density (SPMDD) as confirmed by nuclear densometer testing. Foundation wall backfill should consist of imported free-draining granular material meeting the specifications for OPSS Granular B, or an approved equivalent, compacted to 98 percent SPMDD, taking care to keep heavy compaction equipment from damaging the walls.

The backfill material, if any, in the upper 300 mm below the pavement subgrade elevation should be compacted to 100 percent SPMDD in all areas.

4.9 Buried Utilities

Trench excavations should generally consider Type 3 soil conditions which require side slopes no steeper than 1H:1V to the bottom of the excavation. Where very loose or very soft to soft soils are encountered during excavations, trench slopes should generally consider Type 4 soil conditions which require side slopes no steeper than 3H:1V to the bottom of the excavation. The bedding and cover material for any buried utilities should consist of OPSS 1010 Granular A or B Type II, placed in accordance with pertinent Ontario Provincial Standard Drawings (OPSD 802.013). The bedding and cover material shall be placed in maximum 200 mm thick lifts and should be compacted to at least 98% of SPMDD. The cover material shall be a minimum of 300 mm over the top of the pipe and compacted to 98% of SPMDD, taking care not to damage the utility pipes during compaction.

If wet or saturated conditions exist within any utility excavation, consideration should be given to using 19 mm diameter crushed clear stone wrapped in a geotextile filter fabric as pipe bedding.

4.10 Preliminary Pavement Design

The performance of the pavement is dependent upon proper subgrade preparation. All topsoil and organic materials should be removed down to native material and backfilled with approved engineered fill or native material, compacted to 98% of SPMDD. The subgrade should be compacted, proof rolled, and inspected by a Geotechnical Engineer. Any areas where rutting or appreciable deflection is noted should be subexcavated and replaced with suitable fill. The fill should be compacted to at least 98% of SPMDD.

The recommended minimum pavement structure design has been developed for two (2) traffic loading scenario; light duty and heavy duty. The heavy duty design is appropriate for areas where heavy trucks and maintenance vehicles are anticipated to drive while the light duty design is appropriate for areas where no heavy traffic is anticipated. The recommended minimum pavement structure is provided in Table 11.

Table 11 Recommended Pavement Structure

Pavement Layer	Compaction Requirements	Heavy Duty access road	Light Duty (parking lot)
Surface Course Asphalt	OPSS 310	40 mm HL3 or HL4	40 mm HL3 or HL4
Binder Course Asphalt	OPSS 310	90 mm HL8 (2 lifts)	50 mm HL8
Granular Base	100% SPMDD (ASTM-D698)	150 mm OPSS 1010 Granular A	150 mm OPSS 1010 Granular A
Granular Subbase	98% SPMDD (ASTM-D698)	400 mm OPSS 1010 Granular B	300 mm OPSS 1010 Granular B

Material and thickness substitutions must be approved by the Design Engineer.

The thickness of the subbase layer could be increased at the discretion of the Engineer, to accommodate site conditions at the time of construction, including soft or weak subgrade soil replacement.

Compaction of the subgrade should be verified by the Engineer prior to placing the granular fill. Granular layers should be placed in 200 mm maximum loose lifts and compacted to at least 98 percent of SPMDD (ASTM D698) standard. The granular materials specified should conform to OPSS standards, as confirmed by appropriate materials testing. The final asphalt surface should be sloped at a minimum of 2 percent to shed runoff.

4.11 Design Review and Inspections

Cambium should be contacted to review and approve design drawings, prior to tendering or commencing construction, to ensure that all pertinent geotechnical-related factors have been addressed.

Further Cambium should be retained to complete testing and inspections during construction operations to examine and approve subgrade conditions, placement and compaction of fill materials, granular base courses, and asphaltic concrete.

5.0 CLOSING

Please note that this report is governed by the attached qualifications and limitations. If you have questions or comments regarding this document, please do not hesitate to contact the undersigned at 905-725-6280.

Cambium Inc.

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Qualifications and Limitations

Limited Warranty

In performing work on behalf of a client, Cambium relies on its client to provide instructions on the scope of its retainer and, on that basis; Cambium determines the precise nature of the work to be performed. Cambium undertakes all work in accordance with applicable accepted industry practices and standards. Unless required under local laws, other than as expressly stated herein, no other warranties or conditions, either expressed or implied, are made regarding the services, work or reports provided.

Reliance on Materials and Information

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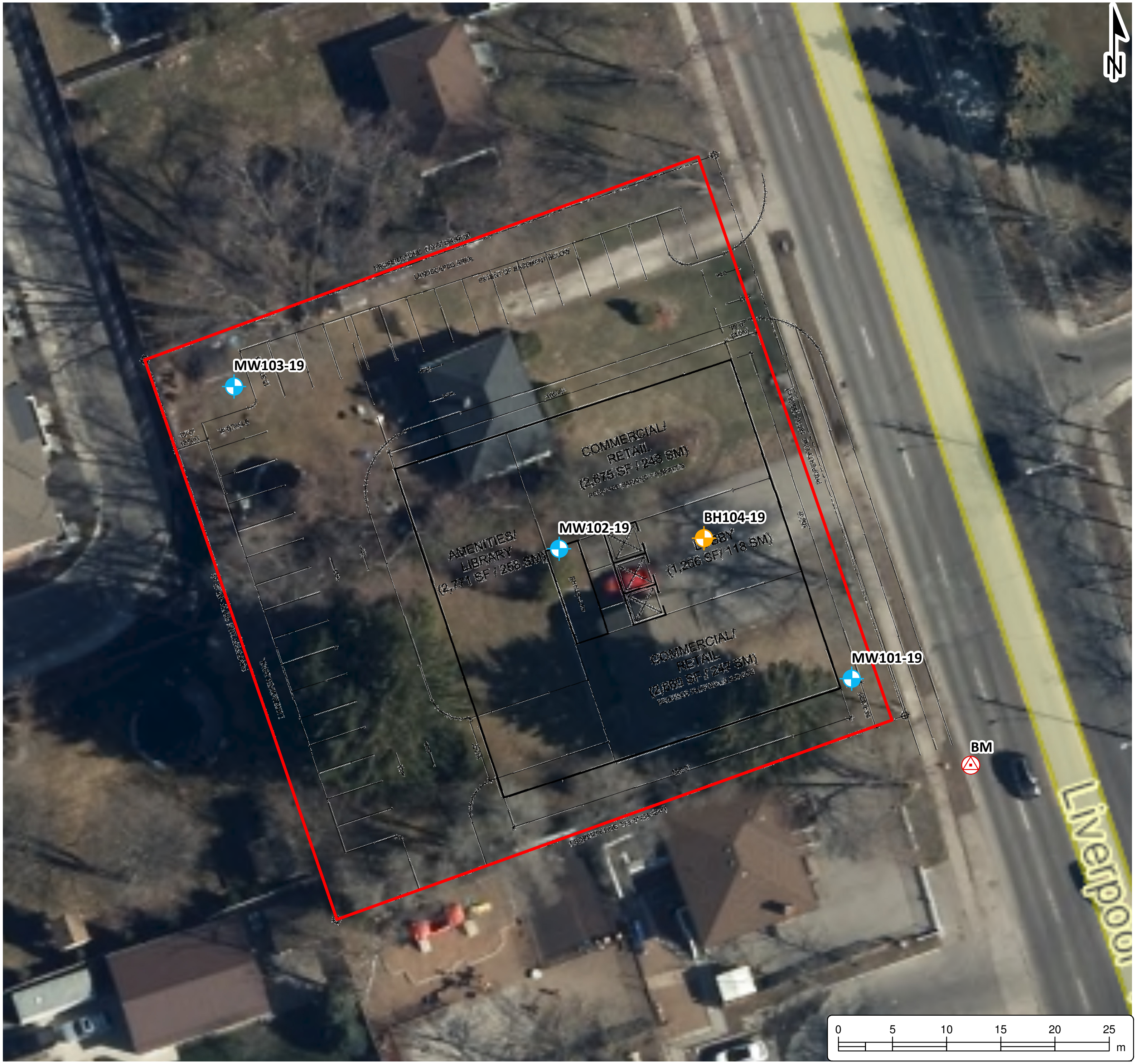
Appended Figures



GEOTECHNICAL INVESTIGATION - ENGINEERING ANALYSIS
GRANT MORRIS ASSOCIATES LTD
1854 Liverpool Road,
Pickering, Ontario

LEGEND

- Benchmark
- Borehole
- Monitoring Well
- Subject Property

Notes:
- Base mapping features are © Queen's Printer of Ontario, 2019 (this does not constitute an endorsement by the Ministry of Natural Resources or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

Benchmark:
BM - Top nut of a fire hydrant located on the west side of Liverpool Road, to the south-west of the subject property.

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BOREHOLE/MONITORING WELL LOCATION PLAN

Project No.: 8960-001		Date: July 2019	Rev.:
Scale: 1:500		Projection: NAD 1983 UTM Zone 17N	
Created by: MAT	Checked by: ZL	Figure: 1	

O:\GIS\project_MXDs\8900 to 8999\8960-001 Grant Morris Associates Ltd. - Geo-Environmental Studies - 1854 Liverpool Road\2019-07-12 FIG 1 Borehole Location Plan.mxd

Appendix A
Borehole Logs

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Log of Borehole: MW101-19

Client:	Grant Morris Associates Ltd.	**Project Name:**	Geotechnical Drilling	**Project No.:**	8960-001
Contractor:	Drilltech	**Method:**	Solid Stem Auger	**Date Completed:**	June 6, 2019
Location:	1854 Liverpool Road	**UTM:**	17 T 653446 m E 4855555 m N	**Elevation:**	89.32 masl

SUBSURFACE PROFILE

Description
TOPSOIL: 330 mm thickness
SILT AND CLAY: Light brown, silt and clay, trace sand, trace organics, about plastic limit, firm
-no organics, stiff
SILTY CLAY: Light brown, silty clay, about plastic limit, firm
-some sand, trace gravel, about plastic limit, stiff
SILT AND CLAY: Grey, silt and clay, trace sand, trace gravel, about plastic limit, very stiff
SILTY CLAY: Light brown, silty clay, some sand, wetter than plastic limit, very soft to soft
SAND AND SILT: Grey, sand and silt, trace clay, trace gravel, saturated, very loose
SILTY SAND: Grey, silty sand, some gravel, some clay, wet, very dense
-trace gravel
Borehole terminated at 14.2 mbgs in silty sand

SAMPLE

Number	Type	% Recovery	SPT (N) / DCPT
1A	SS	83	3
1B			
2	SS	87	9
3	SS	100	10
4	SS	63	6
5	SS	83	12
6	SS	58	17
7	SS	0	2
8	SS	17	1
9	SS	25	1
10	SS	29	4
11	SS	54	>50
12	SS	67	49

Elevation (m) / Depth; Lithology; % Moisture: 25 50 75; SPT (N) / DCPT: 10 20 30 40

Well Installation

Cap
Monument
Bentonite Plug
PVC Standpipe
Sand Pack
PVC Screen
Cap

Remarks

GSA SS3: 1% Sand, 56% Silt, 43% Clay

Water level measured at 3.3 mbgs on June 13, 2019

Water level at 4.0 mbgs upon completion
Borehole cave to 4.2 mbgs upon completion

First groundwater encounter at 6.1 mbgs

GSA SS9: 4% Gravel, 54% Sand, 37% Silt, 5% Clay

Logged By: K.Lewis **Input By:** J.Kimball

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Oshawa
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Log of Borehole: MW102-19

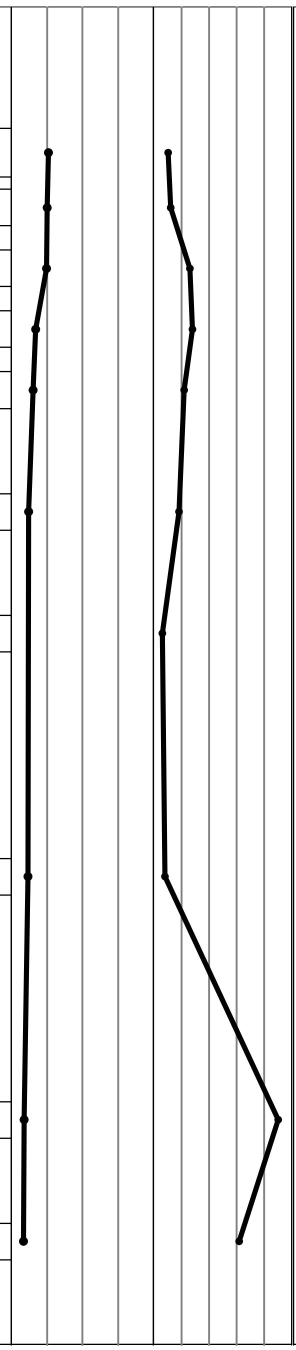
Client:	Grant Morris Associates Ltd.	**Project Name:**	Geotechnical Drilling	**Project No.:**	8960-001
Contractor:	Drilltech	**Method:**	Solid Stem Auger	**Date Completed:**	June 6, 2019
Location:	1854 Liverpool Road	**UTM:**	17T 653389 m E 4855567 m N	**Elevation:**	89.10 masl

SUBSURFACE PROFILE / **SAMPLE**

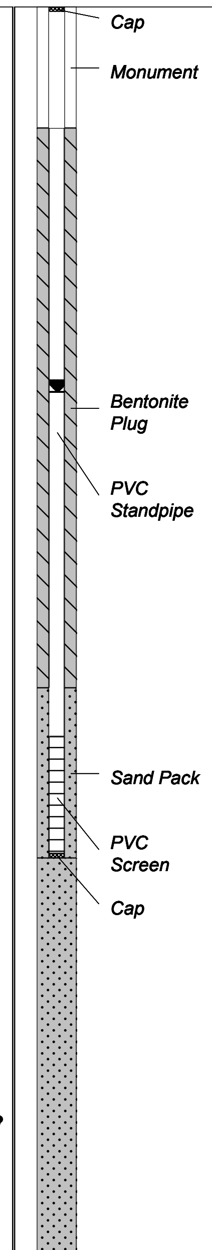
Description	Number	Type	% Recovery	SPT (N) / DCPT
TOPSOIL: 250 mm thickness	1	SS	83	5
SILT AND CLAY: Light brown, silt and clay, trace sand, trace gravel, trace organics, about plastic limit, firm	2	SS	71	6
SILTY CLAY: Light brown, silty clay, about plastic limit, firm	3	SS	96	13
-becomes stiff	4	SS	63	14
SANDY SILT: Light brown, sandy silt, trace clay, trace gravel, moist, compact	5	SS	50	11
SILT AND CLAY: Grey, silt and clay, some sand, trace gravel, about plastic limit, stiff	6	SS	46	9
SILTY SAND: Grey, silty sand, some gravel, some clay, wet, very loose	7	SS	0	3
	8	SS	25	4
-becomes dense	9	SS	54	45
-more clay and gravel	10	SS	33	31
Borehole terminated at 14.2 mbgs in silty sand				

Elevation (m): 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80, 79, 78, 77, 76, 75, 74
Depth: -1, 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

% Moisture: 25 50 75 — SPT (N) / DCPT: 10 20 30 40



Well Installation



Remarks

Water level at 2.7 mbgs upon completion
Water level measured at 3.0 mbgs on June 13, 2019

Borehole cave to 3.0 mbgs upon completion
First groundwater encounter at 3.4 mbgs

GSA SS10: 16% Gravel, 41% Sand, 33% Silt, 10% Clay

Logged By: S.Neumann **Input By:** J.Kimball

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Log of Borehole: MW103-19

Client: Grant Morris Associates Ltd.
Project Name: Geotechnical Drilling
Project No.: 8960-001
Contractor: Drilltech
Method: Solid Stem Auger
Date Completed: June 7, 2019
Location: 1854 Liverpool Road
UTM: 17T 653389 m E 4855582 m N
Elevation: 88.43 masl

SUBSURFACE PROFILE / SAMPLE

Description	Number	Type	% Recovery	SPT (N) / DCPT	Remarks
TOPSOIL: 150 mm thickness	1	SS	38	4	
SILT AND CLAY: Light brown, silt and clay, trace gravel, trace organics, about plastic limit, soft to firm	2	SS	92	5	GSA SS2: 1% Sand, 62% Silt, 37% Clay
SILTY CLAY: Light brown, silty clay, about plastic limit, firm	3	SS	83	10	
-becomes light brown to grey, stiff	4	SS	63	5	Water level measured at 2.3 mbgs on June 13, 2019
CLAYEY SILT: Light brown, clayey silt, some sand, trace gravel, about plastic limit, firm	5	SS	54	11	Borehole caving and water level at 3.7 mbgs completion
GRAVELLY SAND: Grey, gravelly sand, some silt, trace clay, wet, compact; -some gravel	6	SS	54	28	GSA SS6: 26% Gravel, 50% Sand, 15% Silt, 9% Clay
CLAYEY SILT: Grey, clayey silt, some sand, trace gravel, wetter than plastic limit, soft	7	SS	0	5	
SANDY SILT: Grey, sandy silt, some clay, trace gravel, moist, dense	8	SS	63	46	
-becomes compact	9	SS	38	20	
SILTY SAND: Grey, silty sand, some gravel, some clay, moist, very dense	10	SS	50	>50	
Borehole terminated at 12.6 mbgs in silty sand					

Well Installation: Cap; Monument; Bentonite Plug; PVC Standpipe; Sand Pack; PVC Screen; Cap

Logged By: S.Neumann **Input By:** J.Kimball

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Log of Borehole: BH104-19

Client: Grant Morris Associates Ltd. **Project Name:** Geotechnical Investigation **Project No.:** 8960-001

Contractor: Drilltech Drilling Ltd. **Method:** Solid Stem Auger **Date Completed:** July 3, 2019

Location: 1854 Liverpool Road, Pickering **UTM:** 17 T 653432 m E 4855567 m N **Elevation:** 89.50 masl

Depth (m)	Description	Number	Type	% Recovery / TCR (%)	SPT (N) / RQD (%)	Remarks
12.5–14.0	TILL: Dark grey, sandy silt, trace to some gravel and clay, compact, wet	1	SS	58	19	
14.0–16.1	TILL: Dark grey, sandy gravelly silt, dense, saturated	2	SS	100	50/ 75	
16.1–22.2	SHALE: Shale, highly weathered, interbeds of sand and silt, wet	3	SS	100	50/ 75	
	-Interbedded with silt	4	SS	100	50/ 50	Refusal at 21.9 mbgs
22.2–23.5	Shale and Limestone Interbeds: Dark grey to black, shale interbedded with limestone bedrock, very fine grained with thin beds of fine grained, thin to medium bedding, narrowly seperated fractures are smooth to slightly rough	5	RC	74	57	RQD: 57%
	Borehole terminated at 23.5 mbgs on limestone bedrock					

Logged By: J. Monroy **Input By:** R. Mahoney

Appendix B
Physical Laboratory Testing Results

Grain Size Distribution Chart

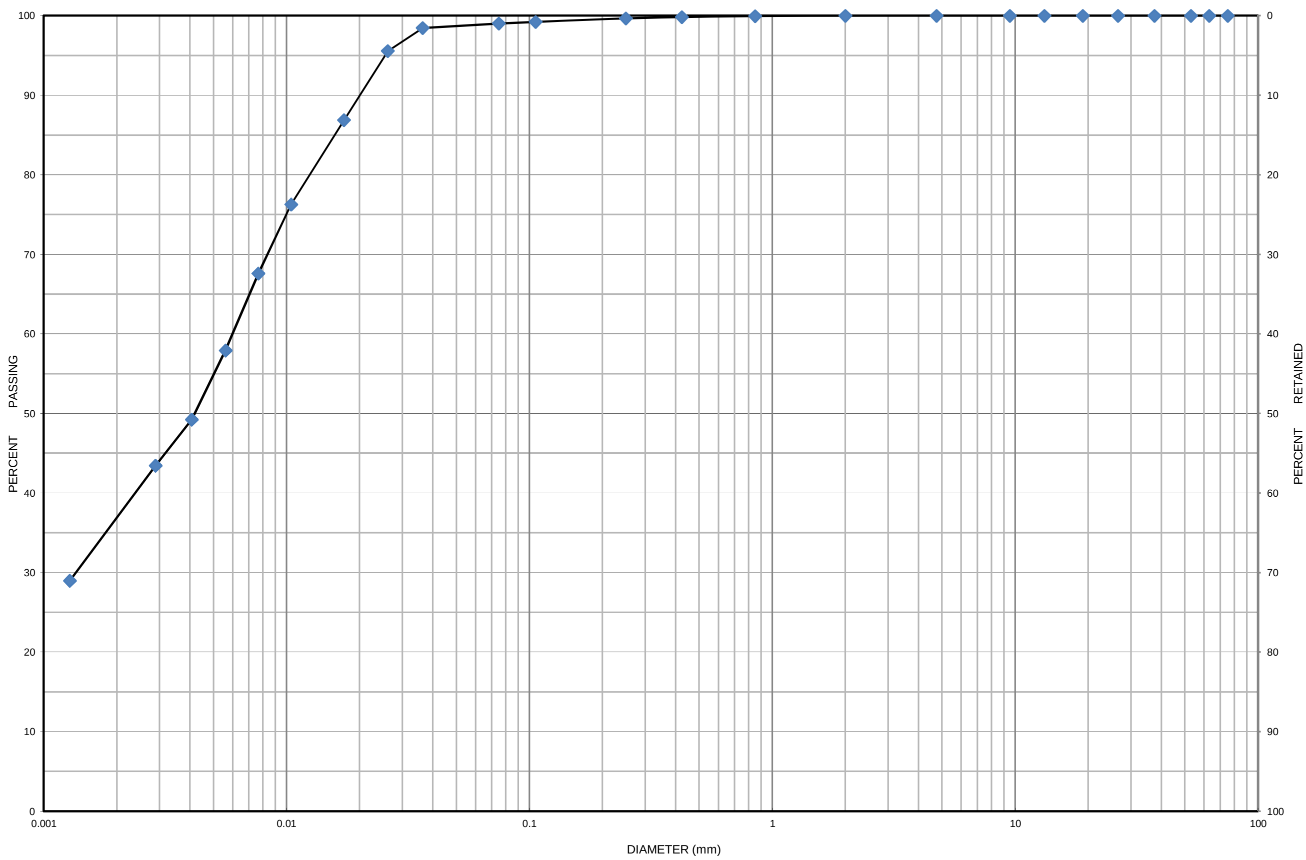
Project Number: *8960-001* **Client:** *Alireza Adejani*

Project Name: *1854 Liverpool Road*

Sample Date: *June 7, 2019* **Sampled By:** *Sean Neumann - Cambium Inc.*

Location: *BH 103-19 SS 2* **Depth:** *0.8 m to 1.2 m* **Lab Sample No:** *S-19-0493*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 103-19	SS 2	0.8 m to 1.2 m	0	1	99		23.2

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silt and Clay trace Sand	ML-CL	0.006	0.0014	-	-	-

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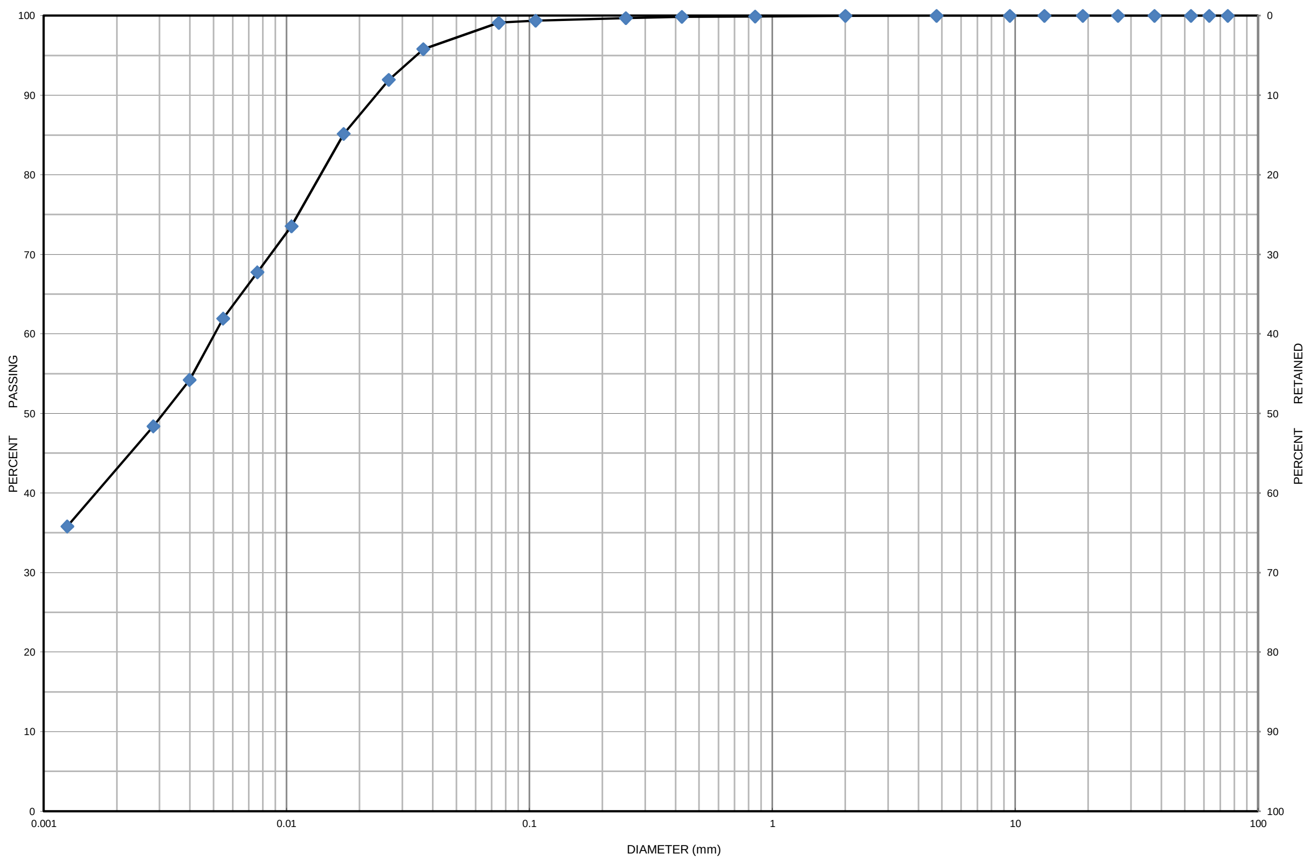
Form: L6V.2 - Grad.Hydo

Grain Size Distribution Chart

Project Number:	*8960-001*	**Client:**	*Alireza Adejani*		
Project Name:	*1854 Liverpool Road*				
Sample Date:	*June 7, 2019*	**Sampled By:**	*Sean Neumann - Cambium Inc.*		
Location:	*BH 101-19 SS 3*	**Depth:**	*1.5 m to 2 m*	**Lab Sample No:**	*S-19-0490*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 101-19	SS 3	1.5 m to 2 m	0	1	99		23.2

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silt and Clay trace Sand	ML-CL	0.005	-	-	-	-

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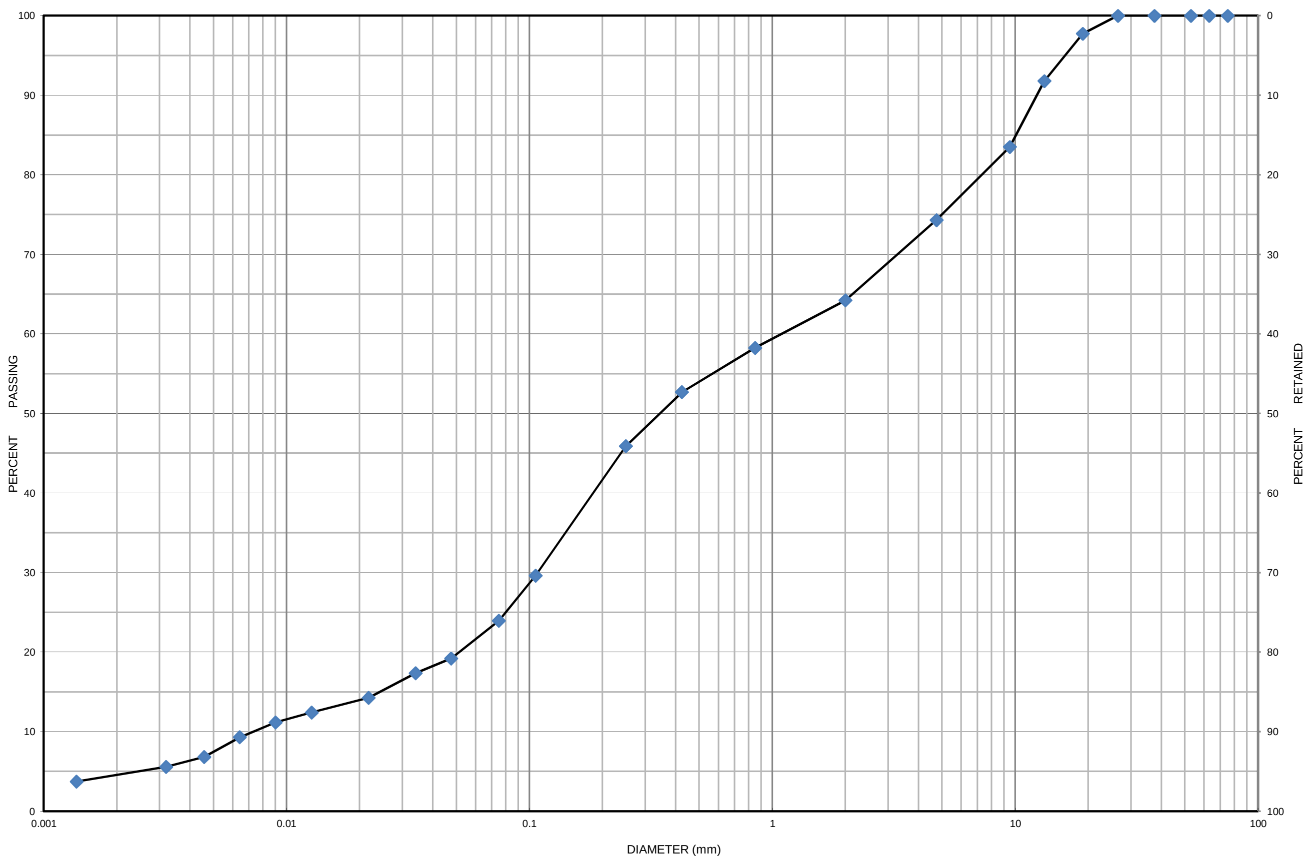
Grain Size Distribution Chart

Project Number: *8960-001* **Client:** *Alireza Adejani*
Project Name: *1854 Liverpool Road*
Sample Date: *June 7, 2019* **Sampled By:** *Sean Neumann - Cambium Inc.*
Location: *BH 103-19 SS 6* **Depth:** *4.6 m to 5 m* **Lab Sample No:** *S-19-0494*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 103-19	SS 6	4.6 m to 5 m	26	50	24		9.7

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Gravelly Sand some Silt trace Clay	SW	1.200	0.120	0.0073	164.38	1.64

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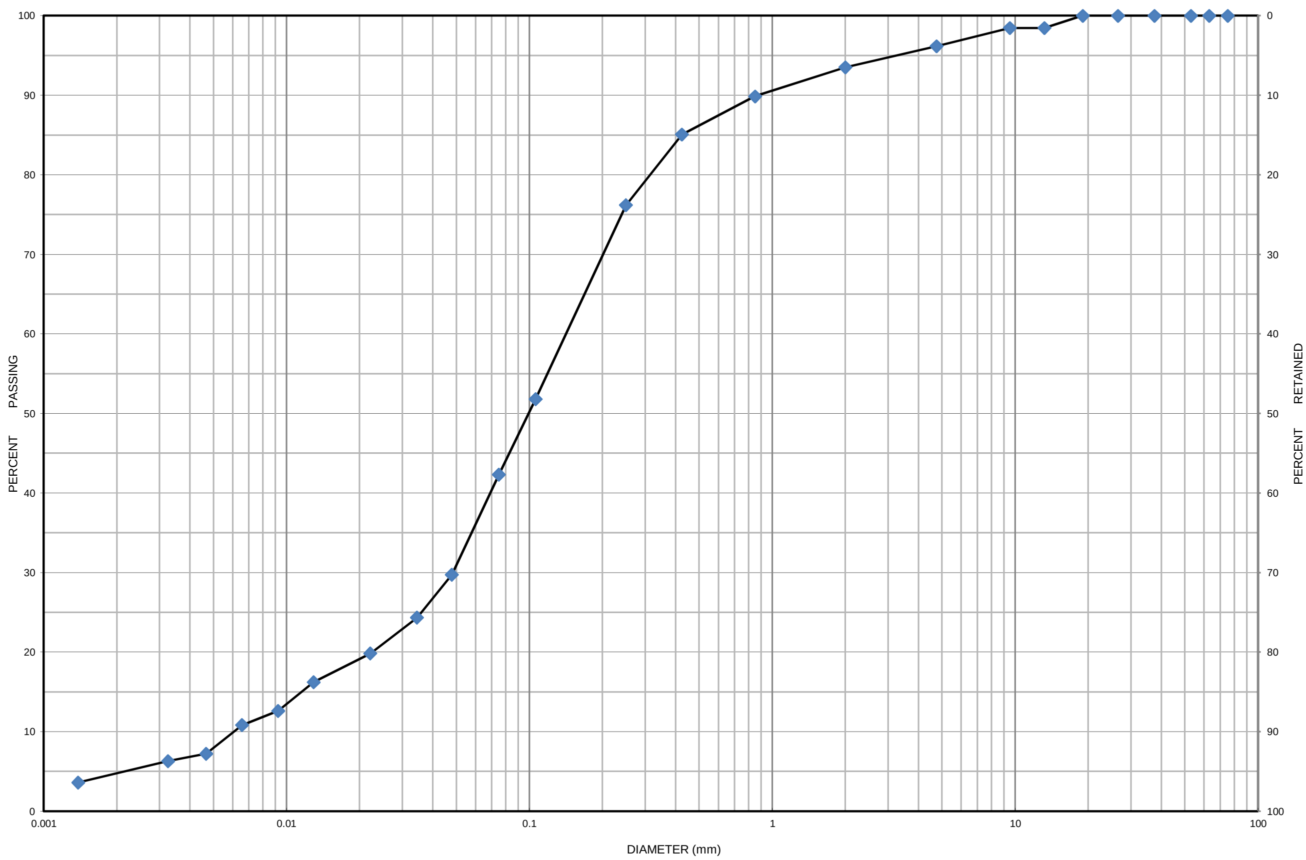
Grain Size Distribution Chart

Project Number:	*8960-001*	**Client:**	*Alireza Adejani*		
Project Name:	*1854 Liverpool Road*				
Sample Date:	*June 7, 2019*	**Sampled By:**	*Sean Neumann - Cambium Inc.*		
Location:	*BH 101-19 SS 9*	**Depth:**	*9.1 m to 9.6 m*	**Lab Sample No:**	*S-19-0491*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH 101-19	SS 9	9.1 m to 9.6 m	4	54	42		12.0

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sand and Silt trace Clay trace Gravel	SW-ML	0.150	0.048	0.0061	24.59	2.52

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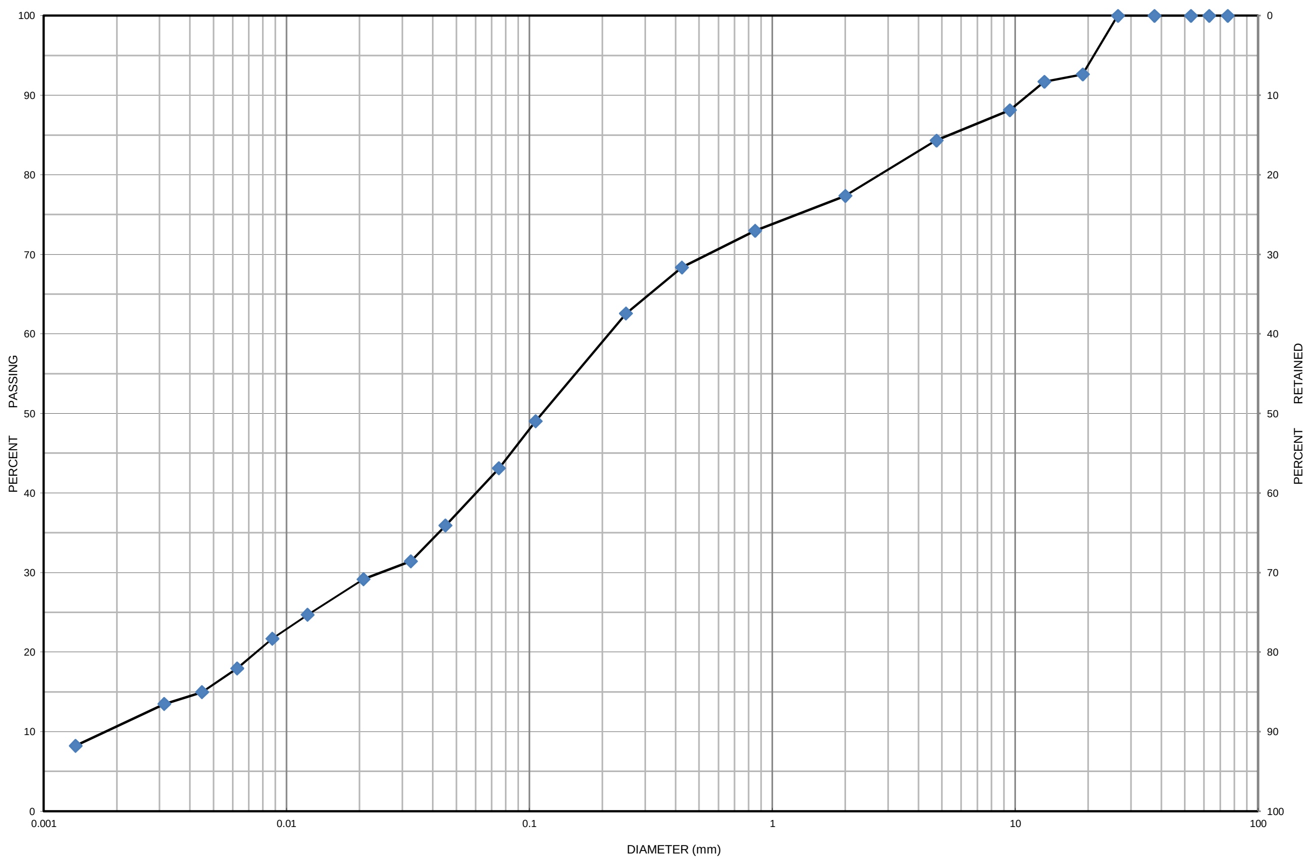
Grain Size Distribution Chart

Project Number: *8960-001* **Client:** *Alireza Adejani*
Project Name: *1854 Liverpool Road*
Sample Date: *June 7, 2019* **Sampled By:** *Sean Neumann - Cambium Inc.*
Location: *BH102-19 SS10* **Depth:** *13.7 m to 14.2 m* **Lab Sample No:** *S-19-0492*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
BH102-19	SS10	13.7 m to 14.2 m	16	41	43		8.0

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silty Sand some Gravel some Clay	SM	0.220	0.024	0.0018	122.22	1.45

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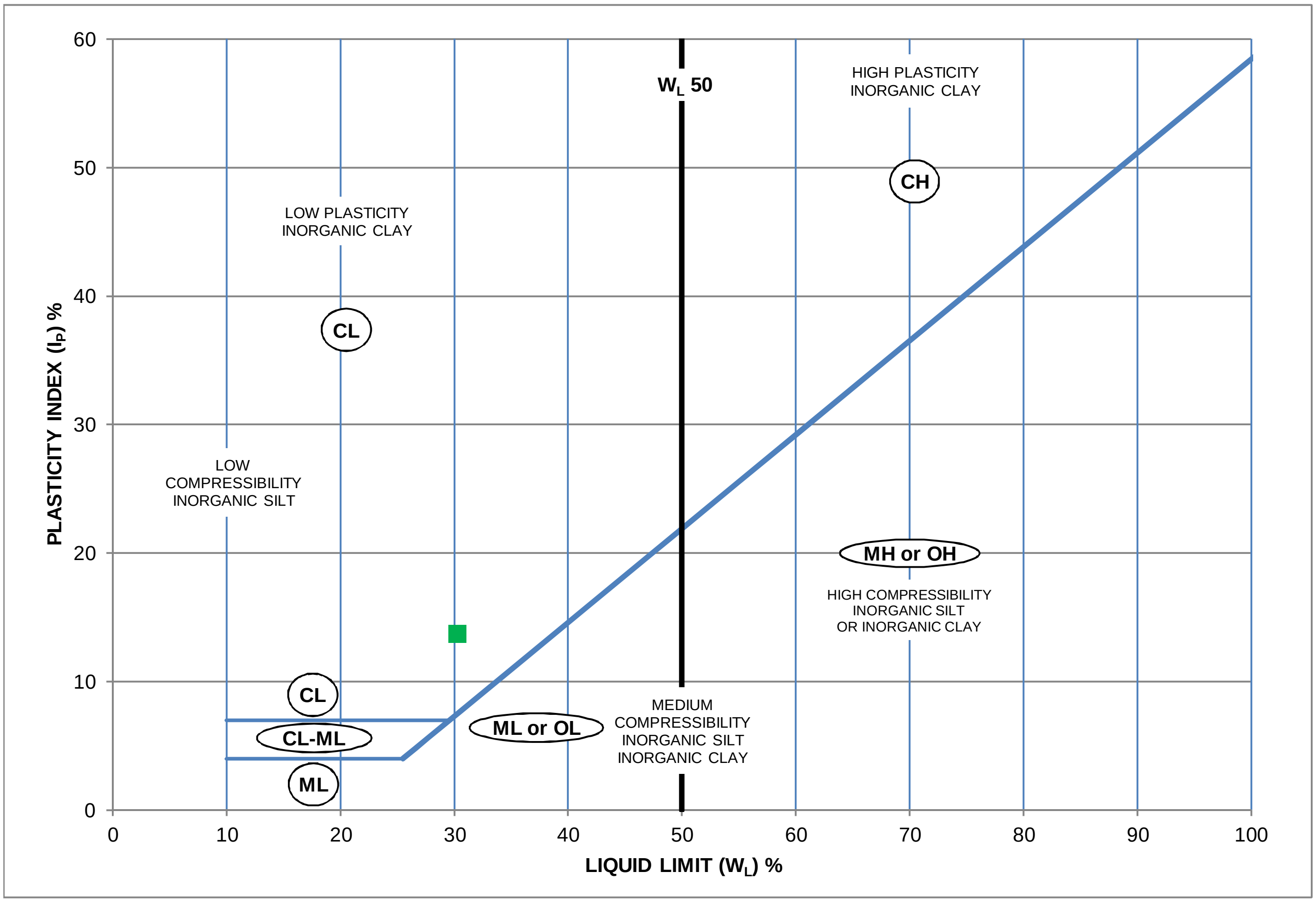
Plasticity Chart

Project Number:	*8960-001*	**Client:**	*Alireza Adejani*
Project Name:	*1854 Liverpool Road*		
Sampled By:	*Sean Neumann - Cambium Inc.*	**Sample Date:**	*June 7, 2019*

Hole No.: *BH 103-19 SS 2* **Depth:** *0.8 m to 1.2 m* **Lab Sample No:** *S-19-0493*

Low Plasticity	High Plasticity



Symbol	Borehole	Sample	Depth	Description
■	BH 103-19	SS 2	0.8 m to 1.2 m	Silt and Clay trace Sand

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
30.3	***16.7***	***13.6***

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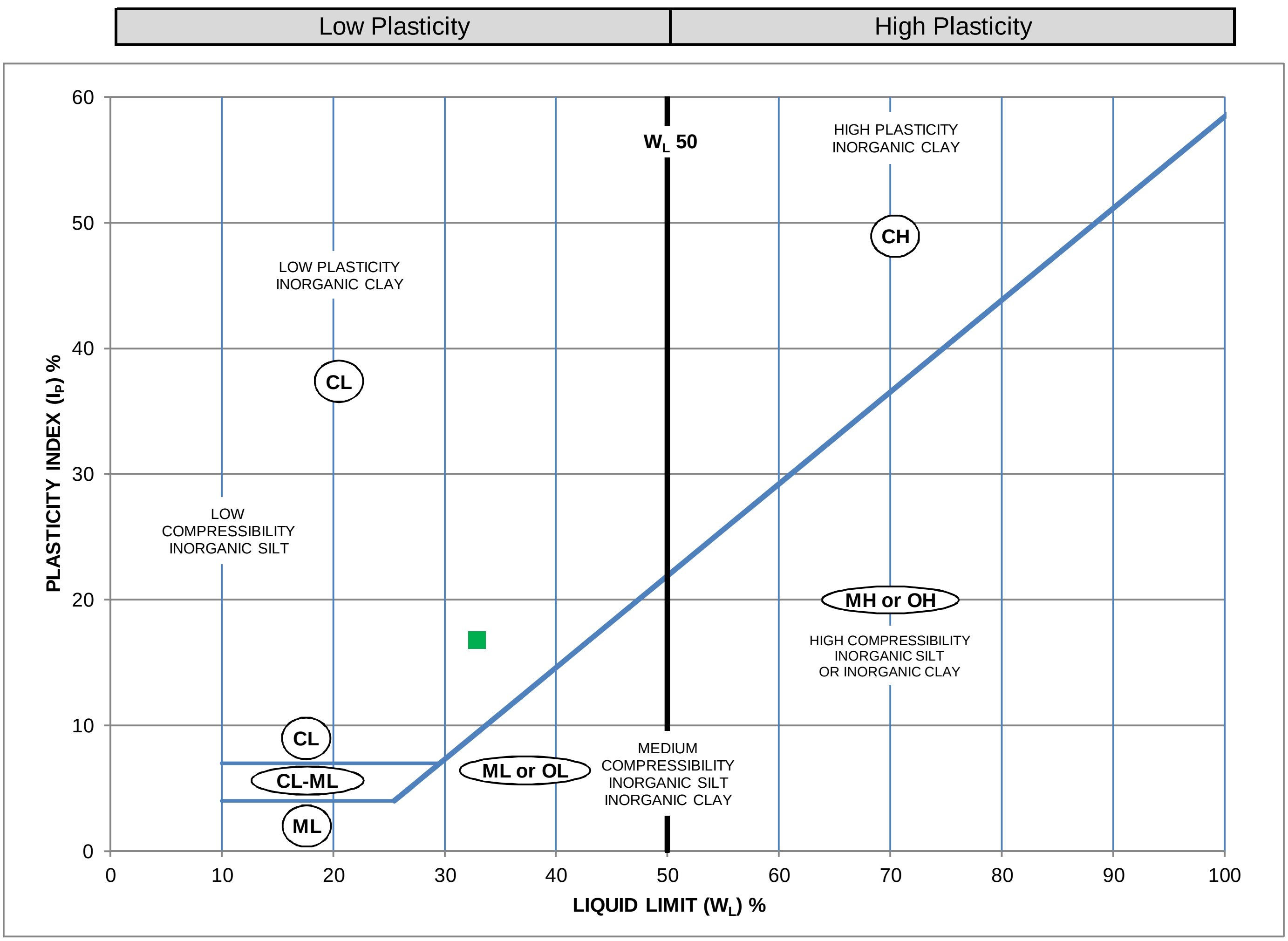
(Senior Project Manager)

Plasticity Chart

Project Number: 8960-001 **Client:** Alireza Adejani
Project Name: 1854 Liverpool Road
Sampled By: Sean Neumann - Cambium Inc. **Sample Date:** June 7, 2019

Hole No.: BH 101-19 SS 3 **Depth:** 1.5 m to 2 m **Lab Sample No:** S-19-0490



Symbol	Borehole	Sample	Depth	Description
■	BH 101-19	SS 3	1.5 m to 2 m	Silt and Clay trace Sand

Liquid Limit (%)	Plastic Limit	Plasticity Index (%)
33.0	16.2	16.7

Issued By: (signature) *Date Issued:* July 16, 2019

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Appendix C

Bedrock Core Photographs

BH104-19:

Photo 1: BH104-19 (22.0 m to 22.5 m depth)

Photo 2: BH104-19 (22.2 m to 22.7 m depth)

Photo 3: BH104-19 (22.5 m to 23.2 m depth)

Photo 4: BH104-19 (23.0 m to 23.5 m depth)