Geotechnical Investigation

97 Bower Street

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1.0 Introduction

SLR (formerly Palmer) was retained by 759509 Ontario Inc. (The Client) to undertake a geotechnical investigation in support of a proposed development application at 97 Bower Street in Acton, Ontario.

It is understood that the development is proposed to consist of a mid-rise building (6-8 stories) with 1 level of underground parking.

The objective of this geotechnical investigation was to determine the subsurface conditions across the site by means of ten (10) exploratory boreholes drilled April 30 to May 2, 2024.

The report is provided on the basis of the terms of reference presented above, and on the assumption that the design will be in accordance with applicable codes and standards. If there are any changes in the design features relevant to the geotechnical analyses, or if any questions arise concerning the geotechnical aspects of the codes and standards, this office should be contacted to review the changes. It may then be necessary to carry out additional borings and reporting before the recommendations of this office can be relied upon.

Laboratory testing for most part follows ASTM or CSA Standards or modifications of these standards that have become standard practice.

This report has been prepared for 759509 Ontario Inc., and their designers. Use of this report by third party without SLR's consent is prohibited. The limitations of the report presented within form an integral part of the document and they must be considered in conjunction with this report.

2.0 Site and Regional Geology

The project limits consist of a currently vacant property located at 97 Bower St. two sections of Wyecroft Road in Halton Ontario. The study area in is situated within the Horseshoe Moraines physiographic region of Southern Ontario (Chapman and Putnam, 1984).

The surficial geology from review of available Ontario surficial geology mapping (Ontario Geological Survey, 2010) shows that the study area is characterized by till moraines, and bedrock geology in the area consists of sandstones, shales and dolostone of the Armabel formation as taken from available mapping (Ontario Geological Survey, 2011).

3.0 Field and Laboratory Work

The field work for the geotechnical investigation was carried out from April 30 to May 2, 2024 by drilling specialists subcontracted to SLR, during which time ten (10) boreholes (BH24-1 to BH24-10) were advanced. The locations of boreholes are shown on the Borehole/Monitoring Well Location Plan, **Drawing 1**. The boreholes were drilled to depths ranging from 5.3 to 15.4 m below existing ground surface.

The boreholes were advanced with a power auger drilling machine, where soil stratigraphy was recorded by observing the quality and changes of augered materials which were retrieved from the boreholes, and by sampling the soils at regular intervals of depth using a 50 mm O.D. split spoon sampler, in accordance with the Standard Penetration Test (SPT) method (ASTM D 1586). This sampling method recovers samples from the soil strata, and the number of blows required to drive the sampler 300 mm depth into the soil (SPT 'N' values) gives an indication of the compactness condition or consistency of the sampled soil material. The SPT 'N' values are indicated on the borehole logs (Refer to **Appendix A**). The field work for this investigation was

supervised by SLR engineering staff, who also logged the boreholes and cared for the recovered samples.

Eight (8) monitoring wells were installed in Boreholes BH25-1 to BH25-8 to determine stabilized groundwater levels. The remaining boreholes without monitoring wells installed were backfilled and sealed upon completion of drilling. The stabilized groundwater levels were measured on May 9, 2024. The monitoring wells installation details and the measured groundwater levels are summarized in **Table 1** and shown in the individual borehole logs.

All soil samples obtained during this investigation were brought to our laboratory for further examination. These samples will be stored for a period of three (3) months after the day of issuing the draft report, after which time they will be discarded unless SLR is advised otherwise in writing. In addition to visual examination in the laboratory, all soil samples from geotechnical boreholes were tested for moisture contents. Grain size analysis of eight (8) soil samples were selected for laboratory testing. The testing results are displayed in **Appendix B** and on the individual borehole logs.

The approximate elevations at the as-drilled borehole locations were surveyed using differential GPS unit. The elevations at the as-drilled borehole locations were not provided by a professional surveyor and should be considered as approximate. Contractors performing the work should confirm the elevations prior to construction. The locations plotted on **Drawing 1** were based on the survey and should be considered as approximate.

4.0 Soil Conditions

4.1 Topsoil

A 60 to 100 mm thick layer of surficial topsoil was encountered surficially in the boreholes. It should be noted that the thickness of the topsoil recorded may not be representative for the site and should not be relied on to calculate the amount of topsoil at the site.

4.2 Fill Materials

Fill materials consisting of sandy silt, silty sand, and sand and gravel were encountered beneath the topsoil. The fill materials extended to depths from about 2.3 to 4.9 m below existing ground surface (Elev. 345.6 to 348.3) in Boreholes BH24-1 to BH24-4, and BH24-6 to BH24-10. The fill extended to termination depth of 5.3 (Elev. 345.1) in Borehole BH24-5. Standard penetration test (SPT) ‘N’ values ranging from 3 to 25 blows per 300 mm penetration indicated a very loose to compact compactness condition. The in-situ moisture contents measured in the fill samples ranged from approximately 6% to 22%.

Grain size analyses were conducted on five (5) selected samples (BH24-2/SS3, BH24-3/SS6, BH24-4/SS4, BH24-5/SS6, and BH24-6/SS5) from the fill materials. The results are presented on the borehole logs and in **Appendix B**, with the following fractions:

Gravel: 0 to 23%

Sand: 25 to 77%

Silt: 15% to 60%

Clay: 4% to 9%

4.3 Silt/Sandy Silt

Silt and sandy silt deposits were encountered below the fill materials in all boreholes except BH24-5. The silt and sandy silt deposits extended to depths of 7.2 to 11.7 m below existing ground surface (Elev. 338.8 to 343.4) in boreholes BH24-7, BH24-9, and BH24-10 and extended to the termination depths of 5.3 to 8.2 m (Elev. 341.9 to 346.0) in Boreholes BH24-1 to BH24-4, BH24-6 and BH24-8. The SPT 'N' values ranging from 2 to 24 blows per 300 mm penetration indicated a very loose to compact compactness condition. 'N' values of 55 and 31 blows per 300 mm were recorded locally in BH24-7 and BH24-10. The natural moisture contents measured in the soil samples ranged from approximately 16% to 27%.

Grain size analyses were conducted on two (2) selected samples (BH24-1/SS5 and BH24-8/SS36) from the silt and sandy silt deposits. The results are presented on the borehole logs and in **Appendix B**, with the following fractions:

Gravel: 0 and 4%

Sand: 5 and 45%

Silt: 48 and 91%

Clay: 3 and 4%

4.4 Sandy Silt Till

Sandy silt till deposit was encountered below the silt deposit in Boreholes BH24-7 and BH24-9. The sandy silt till deposit extended to depths of 8.2 and 14.6 m below existing ground surface (Elev. 342.3 and 335.8). Borehole BH24-9 was terminated in this deposit. The SPT 'N' values ranging from 22 to over 50 blows per 300 mm penetration indicated a compact to very dense compactness condition. The natural moisture contents measured in the soil samples ranged from approximately 11% to 18%.

Grain size analysis was conducted on one (1) selected sample (BH24-7/SS12) from the sandy silt till deposit. The results are presented on the borehole logs and in **Appendix B**, with the following fractions:

Gravel: 6%

Sand: 43%

Silt: 44%

Clay: 7%

4.5 Clayey Silt Till/Shale Complex

Cohesive, clayey silt till/shale complex deposit was encountered below the clayey silt till deposit in Borehole BH24-10 and below the sandy silt till deposit in Borehole BH24-7. The clayey silt till/shale complex extended to termination depths of 15.3 and 15.4 m below existing ground surface (Elev. 335.0 and 335.1). The SPT 'N' values of over 50 blows per 300 mm penetration indicated a hard consistency. The natural moisture contents measured in the soil samples ranged from approximately 8% to 11%.

4.6 Groundwater Conditions

A total of eight (8) monitoring wells (50 mm dia.) were installed to monitor groundwater levels. The stabilized groundwater levels were measured on May 9, 2024. The monitoring well installation details and the measured groundwater levels are summarized in **Table 1** below and shown in the individual borehole logs.

Table 1: Monitoring Well Details and Water Levels

Monitoring Well ID	Screen Interval (mBGS*)	Water Level Depth (mBGS)/ Water Level Elevation (m)
		May 9, 2025
BH24-1	2.0 ~ 5.0	1.4 / 349.2
BH24-2	2.0 ~ 5.0	1.2 / 349.3
BH24-3	2.0 ~ 5.0	1.4 / 349.2
BH24-4	2.0 ~ 5.0	0.9 / 349.2
BH24-5	2.0 ~ 5.0	1.4 / 349.1
BH24-6	2.0 ~ 5.0	3.2 / 348.1
BH24-7	11.5 ~ 14.5	1.2 / 349.2
BH24-8	4.5 ~ 6.0	1.0 / 349.2
*Note: mBGS = meter below ground surface		

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations in response to weather events.

5.0 Discussion and Recommendations

It is understood that the development is proposed to consist of a mid-rise building (5 stories) with 1 level of underground parking. The following paragraphs will discuss the geotechnical investigation results and corresponding recommendations for the foundation. The lowest finished floor elevation (FFE) was not provided at the time of writing this report and for the purposes of writing this report, it is assumed that the FFE will be approximately 347.5 m (3.0 m below the current ground elevation of approximately 350.5 m). Footing bases are anticipated to be constructed at about 1.0 m lower than the lowest FFE (Elev. 346.5).

5.1 Foundation Considerations

5.1.1 General Foundation Considerations

The existing fills and loose/very loose soils found at the anticipated founding level are not suitable for supporting foundations of the proposed structures. With the extensive depths of fill and loose soils, shallow foundations would not be feasible. . Additional borehole investigation is required to verify the suitable native soil deposits for feasible foundation support.

In the vicinity of existing buried utilities, all foundations must be lowered to undisturbed native soils, or alternatively the utilities must be structurally bridged. Where it is necessary to place foundations at different levels, the upper foundation must be founded below an imaginary 10

horizontal to 7 vertical line drawn up from the base of the lower foundation. The lower footing must be installed first to help minimize the risk of undermining the upper foundation.

It should be noted that the recommended bearing resistances have been estimated by SLR from the borehole information for the design stage only. The investigation and comments are necessarily on-going as new information of the underground conditions become available. For example, more specific information is available with respect to conditions between boreholes when foundation construction is underway. The interpretation between boreholes and the recommendations of this report must therefore be checked through field inspections to validate the information for use during the construction stage

5.1.2 Helical Piers

The proposed building may be supported by helical piers founded in the very dense/hard till below elevation of about 338.0 m in the locations of BH24-7 and BH24-10, providing the required pile torque rating is capable of installing piles into the deep founding levels. Possible large obstructions such as buried concrete slabs or existing foundations within the fill materials or boulders within the tills may be encountered during the installation of the helical piers and may prevent the piles from reaching proposed founding depths.

Helical piers/anchors are proprietary products design, supplied and installed by specialist contractors. Bearing capacity and other design details regarding helical piers can be discussed with the specialized contractor. Geotechnical comments concerning installation and design capacities are provided in the following paragraphs. It must be noted that they are considered to be preliminary values suitable for preliminary design only. The actual design and installation of helical piers should be undertaken by contractors in Ontario that are approved by the manufacturer.

The designer should define the depth and type of helical piers according to the soil conditions and the required design loads. The designer should also consider the buckling resistance in weak soils and their lateral capacity. The contractor should also be responsible for the design capacity of the foundation units. In this regard, it is recommended that compression and tension tests be conducted to verify helical pier capacities prior to final design.

It should be noted that there is a possibility that the very dense or hard stratum may not extend uniformly throughout the areas and the need for deeper helical piers in some areas must not be overlooked.

If site grades are raised, consideration should be given to the effects of negative skin friction on the helical piers.

It is recommended that SLR be retained to monitor and document helical pier installation to verify that the recommended capacity is achieved.

5.1.3 Ground Improvement

As an alternative to deep foundation methods, ground improvement techniques such as rammed aggregate piers (RAPs) or controlled modulus columns (CMCs) may be a viable solution to increase bearing capacity for this Site. Utilizing RAPs or CMCs would have several potential benefits for this Site, including higher bearing capacity, increased Seismic Site Class, as well as potentially reducing excess soil and earthworks compared to other deep foundation methods.

A ground improvement will be installed through a granular working platform. The design would include diameter, spacing and depths to provide an improved subgrade designed to support the required footing and floor slab loads and control settlement within tolerable limits.

CMCs are vertical semi-rigid inclusions, i.e. injected columns, with concrete or mortar, installed with a hollow full displacement auger. During the auger extraction process, the column is developed by grouting under controlled limited pressure through the stem of the auger. Column diameter varies between 250 and 450 mm. A load transfer platform is designed between the top of the columns and the structure (slab or foundation) to transfer the load to the CMC. The solution is to provide increased stiffness to the soil mass to safely control both total and differential settlements. The structure's loads are transferred between the soil and the CMC elements.

As this technology is proprietary and performed by specialty contractors, an exact bearing capacity cannot be provided, and a specialty contractor must be retained to design and install ground improvement. The designer of specialty contractors should design the depth and methods of installation according to soil conditions and the required design loads. Further recommendations regarding bearing capacity and other design details regarding RAPs or CMCs can be discussed with the specialty contractor. Detailed design services of ground improvement are available through a number of specialty design-build contractors, and stamped shop drawings should be provided to SLR for review.

5.1.4 Pile Foundations

5.1.4.1 Continuous Flight Auger (CFA) Piles

Considering the predominantly sandy silt /silt deposits underlain by very dense sandy silt till and hard clayey silt till encountered at the Site, the proposed building may be supported by continuous flight auger (CFA) piles founded in the very dense/hard native soils below elevation 338.0 m. CFA piles are cast-in-place concrete piles installed by advancing hollow stem augers. Once the augers have reached the design founding depths, concrete is continuously pumped through the hollow stem augers as they are slowly retracted, matching the rate of concrete pumping.

Should the CFA piles be considered for uplifting resistances, a reduction factor of 0.75 should be considered. A qualified pile design engineer should determine the length and size of the piles, based on the required structural design loads and the subsurface conditions.

It is highly recommended that static load testing of the piles be carried out to confirm the bearing resistance provided. Minimum 2 field load tests per size of CFA piles are recommended to confirm the availability of the assumed bearing values. The test piles must be loaded to at least 2 times the designed ULS bearing resistance. Depending on the load test results, larger/deeper piles may be required to achieve the design bearing resistances. As a lower factor of safety can be used when static load testing results are available, the results of the static load testing may also be used to optimize the design lengths of the foundations. The load testing should be carried out as far in advance of the anticipated start of production piling to permit any required changes in the design of the foundations, if necessary, prior to the production phase of the foundation installation.

The bearing resistances of CFA piles will be highly dependent on the contractor's experience, the quality and procedure of the pile installation, and the skills of the installation operator(s). The CFA contractor must review the borehole information and evaluate bearing capacity of the piles based on their experience. The quality and the design bearing resistance of the piles must be ensured by the CFA contractor. A specialty contractor should be retained to design and install the CFA piles based on the performance specification and design bearing resistances.

Due to the very soft/loose nature of the native soils, there is a potential for "necking" of the CFA piles during installation. An important consideration in CFA pile construction is matching the rate of withdrawal of the augers with the rate of pumping concrete such that "necking" of the pile does

not occur. It is recommended that integrity testing also be carried out as part of the Quality Assurance process for the piling works to confirm that the piles installed have a consistent diameter over their full length. Piles that have installation records (i.e. pumping rate) out of specification or otherwise appear abnormal should be subjected to integrity tests (i.e. sonic echo tests or pile integrity tests) or verification load tests to determine if they should be accepted or rejected

In order to avoid group effect on the bearing capacity of the piles, the horizontal spacing of adjacent piles should be at least 3 times its dimension/diameter. Plumbness and location of the CFA piles should follow the design specification provided by the structural engineer.

5.1.5 Excavation and Backfilling

According to the results of this investigation, the excavations are generally anticipated to be carried out in the fill materials, native sandy silt / silts. Imported backfill may be required in areas where native backfill cannot be compacted to 98% SPMDD. This excavation in the overburden can be carried out with heavy hydraulic backhoe while recommendations for excavations in bedrock are discussed below.

It should be noted that the fill materials are a mixture of cohesionless and cohesive materials and may contain boulders. Possible large obstructions such as buried concrete pieces and existing foundations are also anticipated in the fill material. Provisions must be made in the excavation contract for the removal of possible boulders or obstructions in the fill material.

All excavations must be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). In accordance with OHSA, the fill materials and very loose to loose sandy silt / silts would be classified as Type 3 Soils above the groundwater table and Type 4 Soils below the groundwater table.

It may be necessary to flatten the excavation slopes if seepage zones or loose/soft conditions are encountered locally. Excavations should be continuously examined for any evidence of instability if left open for extended periods of time or following weather events such as rain or freeze/thaw.

It is anticipated that excavations at the Site will consist of temporary open cuts with side slopes not steeper than 1H:1V. However, depending on the construction procedures adopted by the contractor and weather conditions at the time of construction, some local flattening of the slopes might be required. Where side slopes of excavations are to be steepened, then a positive excavation support system should be considered.

Vertical cuts in any of the soil types and oversteepend slopes should be supported with shoring and bracing to safeguard the stability of the sides of the excavation. All shoring designs should be in accordance with the 5th Edition of the Canadian Foundation Engineering Manual and must be reviewed by a professionally qualified geotechnical engineer. If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding as frost will induce an additional load to the shored wall. It should be noted that a trench box may not be utilized for purposes of shoring the excavation as defined in the Occupational Health and Safety Act.

Utilities, settlement sensitive structures and foundations located within close proximity to the excavation may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are presented in **Drawing 4.**

5.2 Temporary Shoring and Shoring Movement Monitoring

In view of the anticipated underground parking structure of the subject buildings and excavation up to an elevation of 346.5.0m and the fact that the proposed structures will extend close to the property lines, it is anticipated that the proposed excavations will be supported by a temporary shoring system. Contiguous cut-off caisson wall may be considered to reduce the groundwater seepage during the temporary dewatering stage. Alternatively, soldier piles with wood lagging could be considered, provided that adequate groundwater control is achieved and the contractor is prepared to minimize the expected caving and groundwater seepage once the excavation extends below the groundwater table and into the sandy silt to silt deposits.

The contiguous concrete caisson walls could be supported by tie-back anchors or struts. For post tensioned pressure-grouted soil anchors installed into firm to stiff soils, the allowable (SLS) bond stress between grout and soil can be assumed to be 50 kPa, but in no case should the bonded length be less than 6m.

The top anchor must not be placed lower than 3.0 meters below the top of ground surface. Casings will be required for the anchors when penetrating through sandy/silty deposits to prevent from soil caving. Bond values are suggested but these values are arbitrary since the contractor's installation procedures will determine the actual soil to concrete bond value. Hence, the contractor must decide on a capacity and confirm its availability. The actual capacity (bond resistance) of the anchors should be established by full scale pull-out tests ("performance test") at each anchor level in accordance with CFEM (5th edition), testing to 200% of working load. Each installed anchor must be proof loaded to 1.33 times the design working load, in accordance with Post-Tensioning Institute (PTI) guidelines. The ground anchors should be double corrosion protected (i.e. PTI Class I). Adhesion on the behind the shoring system must be neglected when designing this shoring system.

An SLS bearing capacity can be taken as 150 kPa for pile/caissons founded in the hard clayey silt till.

Lagging walls are assumed to permit drainage of perched water between the lagging boards. No water pressure has been assumed to act against the shoring for lagging walls which are assumed to permit drainage of perched water between the lagging boards. Construction caisson walls need to be designed for hydrostatic water pressure, taken as 1m higher than the highest measured from nearby monitoring wells. The top of the shaft lining should be raised above the grade in consideration of the risk of shaft flooding. Surcharge load due to construction machinery and traffic must be considered.

If shoring is to be carried out over the winter months or if the excavation is to be left open for any period during below zero temperature, shored walls must be protected against frost penetration by means of insulation or heated hoarding. The soldier piles/caissons should be installed in pre-augered holes drilled into the piles. No loss of ground should be permitted during augering for soldier piles/caissons and the drilling contractor should be warned of the potential for encountering overburden obstructions within the fill and obstructions with the till such as cobbles and/or boulders. The use of liners may be required to keep the augered holes open before concrete placement. In the event that loss of ground occurs during lagging installation, the depth of excavated lifts must be reduced. In critical areas, the lost soils must be replaced by grouting. The concrete strength must be specified by the shoring designer.

For contiguous concrete caisson shoring, the rate of groundwater seepage is expected to be slow to moderate and can be handled by gravity drainage and pumping from filtered sumps established at the base of the excavation.

All shoring designs should be in accordance with the 5th Edition of the Canadian Foundation Engineering Manual and must be reviewed by a qualified geotechnical engineer. The soil parameters estimated to be applicable for this design are provided in **Table 2**.

Monitoring of shored wall deflections by means of survey targets is recommended in areas where settlements could damage existing utilities, infrastructures, or buildings.

Movement of the shoring system is inevitable. Vertical movement will result from the vertical load on the shoring system resulting from the inclined tiebacks and inward horizontal movement results from earth and water pressures. The magnitude of this movement can be controlled by sound construction practices.

To ensure that movements of the shoring are within an acceptable range, a monitoring program must be carried out. Vertical and horizontal targets on the soldier piles must be located and surveyed before excavation begins. Weekly readings during excavation should show that the movements will be within those predicted; if not, the monitoring results will enable directions to be given to improve the shoring. The movement should be monitored throughout the construction period.

5.3 Frost Protection

All foundations exposed to seasonal freezing conditions must have at least 1.4 metres of soil cover for frost protection.

There is no official regulation governing the required soil cover for frost protection of footings below unheated basement floors. Certainly, it will not be greater than the 1.4m required for exterior footings. Unconfirmed experience suggests that shallower depths of soil cover of 0.9m for interior column footings and 0.6m for wall footings have been adequate in case of one (1) level of basement. Adjacent to air shafts and entrance/exit doors, a footing depth of 1.4m below floor level is required or, alternatively, insulation must be provided.

It is also emphasized that underfloor drainage and/or an adequate free draining gravel base is required to minimize the risk of floor dampness. Floor dampness could lead to temporary icing and the risk of accidents.

5.4 Floor Slab and Permanent Drainage

With one level of underground parking, the floor slab can be supported on the undisturbed native soils encountered at the Site. The foundation and underground parking may be designed as a water-tight structure, assuming the hydrostatic water pressure at the ground surface. However, a moisture barrier consisting of at least 300 mm of 19 mm clear crushed stone should be installed under the floor slab, in case minor water seepage may migrate through the waterproof system. Where a raft foundation is used a moisture barrier consisting of a 300 mm thick layer of 19 mm clear crushed stone and subdrains should be installed between the top of the raft and the underside of the floor slab.

If the foundation and underground parking are not designed to be water-tight, a perimeter drainage system will be required. Typical drainage and backfill recommendations for the underground parking structures are illustrated on **Drawings 2** to **4** for open cut and shored excavations. Where the floor slabs are below the prevailing groundwater table, the placement of underfloor drainage should be considered.

Special care should be taken to ensure compaction around columns and adjacent to foundation walls. Unless the foundations are designed to account for the floor slab loads, the floor slabs should be structurally separated from the foundation walls and columns. Sawcut control joints

should be provided at regular intervals and along column lines to minimize shrinkage cracking and to allow for differential settlement of the floor slabs.

Where the backfill against the exterior walls is to support settlement sensitive structures, such as concrete slabs, pavements or walkways, it should be uniformly compacted to at least 98% of SPMDD.

5.5 Elevator Pits

The elevator pits can be designed as water-tight structures, and water pressure on the pit walls and the slab should be taken into consideration by the design engineer, assuming the water table at about 0.3m below the adjacent basement floor if a subfloor drainage system is considered. If there is no subfloor drainage system, the water pressure on the pit wall and the slab should be considered to be at least 1.0m higher than the highest measured groundwater table.

Should it be required by the designer, a drainage system at the base level of the elevator pits may also be considered together with the watertight design of the elevator pits, in case of possible minor water seepage migrating through the waterproof system.

5.6 Lateral Earth Pressure

The lateral earth pressure acting on the permanent rigid walls of the underground structures in overburden soils can be evaluated by the following formula:

$$\mathbf{P_h = K (\gamma\, h + q)}$$

where P_h = Lateral earth pressure acting at depth “h” (kPa)

K = Earth pressure coefficient, assumed to be 0.40 for vertical walls

horizontal ground surface condition for permanent construction

γ = Unit weight of backfill, which can be assumed to be 21 kN/m^3

h = Depth below finished grade of the point of interest (m)

q = Equivalent value of surcharge on the ground surface (kPa)

If there is no provision for drainage of the backfill behind the subsurface walls, the walls should be designed to resist the lateral earth pressure and the hydrostatic pressure imposed by the backfill adjacent to the wall.

Below the water table, the submerged unit weight of the soil should be used and the full hydrostatic water pressure from ground surface should be added. If the ground surface is not horizontal, the uneven portion can be treated as an equivalent surcharge load.

5.7 Seismic Considerations

The 2012 Ontario Building Code (OBC 2012) came into effect on January 1, 2014 and contains updated seismic analysis and design methodology. The seismic site classification methodology outlined in the code is based on the subsurface conditions within the upper 30 m below existing grade.

The conservative site classification is based on physical borehole information obtained at depths of less than 30 m and based on general knowledge of the local geology and physiography. In this regard, SLR’s drilling program included boreholes drilled to depths up to 15.4 m below the

existing ground surface. Based on the borehole information and our local experience, a Site Class E may be used for the design for this site.

Should optimization of the site class be recommended by the structural engineer, a field seismic shear wave velocity test should be considered to confirm the classification.

5.8 Groundwater Control

It is expected that any seepage above the groundwater table can be removed by pumping from sumps in the excavation area. However, significant seepage may be expected once the excavations extend below the prevailing groundwater tables into both the overburden. Depending upon the prevailing groundwater level at the time of construction, “active, advance” dewatering measure using well points/eductors may be required to maintain the stability of the base and side slopes of the excavations in these areas. These “active dewatering” measures would have to be installed and then operated for a week or two in advance of excavation work progressing to these areas. A contractor specializing in dewatering should be retained to design the active dewatering systems.

The presence of surface water runoff and ground water levels may fluctuate subject to seasonal variations and precipitation patterns.

The need for dewatering the trench due to inflow of nearby drainage ditches or surface water after a rainfall event should not be overlooked. The contractor should divert drainage ditches away from the excavation. The need for additional pumps should not be overlooked.

It should be noted that if the construction dewatering system/sumps result in a water taking of more than 50,000 L/day. It is noted that recent changes to Ontario Regulation (O. Reg.) 63/16 under the Environmental Protection Act effective July 1, 2025, removed the maximum daily rate where previously dewatering greater than 400,000 L/day triggered the need for a Permit to Take Water (PTTW).

5.9 Pavements

The recommended pavement structures provided in **Table 5** are based upon borehole information obtained in this investigation. The values may need to be adjusted based on the municipality/regional standards. Consequently, the recommended pavement structures should be considered for reference purposes only. A functional design life of eight to ten years has been used to establish the pavement recommendations. This represents the number of years to the first rehabilitation, assuming regular maintenance is carried out. If required, a more refined pavement structure design can be performed based on specific traffic data and design life requirements and will involve specific laboratory tests to determine frost susceptibility and strength characteristics of the subgrade soils, as well as specific data input from the client.

Table 5: Recommended Pavement Structure Thickness

Pavement Layer	Compaction Requirements	Light Duty Pavement (Parking for Cars)	Heavy Duty Pavement (Access Road, Fire Routes, occasional Delivery Trucks)	Heavy Truck Use Pavement
Asphaltic Concrete	92% Maximum Relative Density (MRD)	40 mm HL 3 50 mm HL 8	40 mm HL 3 80 mm HL 8	40 mm HL 1 120 mm HL 8
OPSS Granular "A" Base (or 20mm Crusher Run Limestone)	100% SPMDD*	150 mm	150 mm	150 mm
OPSS Granular "B" (or 50mm Crusher Run Limestone)	100% SPMDD	250 mm	350 mm	450 mm

* Denotes Standard Proctor Maximum Dry Density, ASTM-D698

The subgrade must be compacted to 98% SPMDD for at least the upper 500 mm unless accepted by SLR.

The pavement design considers that construction will be carried out during the drier time of the year and that the subgrade is stable, as determined by proof-rolling operations. If the subgrade should become excessively wet or rutted during construction activities, additional subbase material may be required. The need for additional subbase is best determined during construction.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure uniform subgrade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be over-emphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped (preferably at a minimum grade of two percent) to provide effective surface drainage toward catch basins. The excavation around catch basins and manholes should be backfilled with free-draining granular material to minimize differential movements between the pavement and structures due to frost action. The manholes/catch basins should be provided with perforated stub drains to permit drainage of the backfill. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas. Subdrains should be installed to intercept excess subsurface moisture and prevent subgrade softening. This is particularly important in heavy-duty pavement areas.

Additional comments on the construction of internal roadways and parking areas are as follows:

1) As part of the subgrade preparation, proposed area for internal roads and pavements should be stripped of topsoil and other obvious deleterious material. Fill required to raise the grades to design elevations should conform to backfill requirements outlined in previous sections of this report. The subgrade should be properly shaped, crowned then proof-rolled in the full-time presence of a representative of this office. Soft or spongy subgrade areas should be

sub-excavated and properly replaced with suitable approved granular backfill compacted to 98% SPMDD.

2) The locations and extent of sub-drainage required within the roadways and other paved areas should be reviewed by a pavement engineer in conjunction with the proposed site grading. The subdrains should be properly filtered to prevent the loss of (and clogging by) soil fines. Assuming that satisfactory crossfalls in the order of two percent have been provided, subdrains extending from and between catch basins may be satisfactory. If shallower crossfalls are considered, a more extensive system of sub-drainage may be necessary and should be reviewed by a pavement engineer.

The most severe loading conditions on light-duty pavement areas and the subgrade may occur during construction. Consequently, special provisions such as restricted access lanes, half-loads during paving, etc., may be required, especially if construction is carried out during unfavourable weather.

5.10 Geotechnical Quality of Excavated Soils

Reference to the borehole logs suggests that the excavated materials with respect to their compaction characteristics can be divided into four groups:

- Group 1 soils comprise the cohesionless to low plasticity sandy silt / silt. The compaction of these soils will require a very tight control of their moisture content during placement and compaction. At moisture contents more than 3% below the optimum, the soil will likely be dusty and “flour” like while at moisture contents ±1% higher than optimum, the soil will be “spongy” and will “pump”.
- Group 2 soils consist of unsuitable materials because of their high moisture, organic inclusions, or deleterious inclusions, including all of the existing fill materials. These soils should be either disposed off-site or should be used only in “soft” landscaping areas where they can be placed with nominal compaction, and where surface settlements are tolerable.

As a general requirement, all backfill material should be placed in 200 to 300mm thick loose lifts and compacted to at least 96% of SPMDD, at a placement moisture content within ±2% of the optimum. Below existing/future roads, the backfill must be Granular “A” or “B” material, and the top 1.5m of subgrade backfill below the underside of the pavement structure should be compacted to 98% of SPMDD.

6.0 Closure

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

This report was prepared, reviewed and approved by the undersigned.

Regards,

SLR Consulting (Canada) Ltd.



Sam MacDonald, EIT
Geotechnical Project Manager

Chi Tseng (Dennis) Tseng, M.Sc., P.Eng.
Principal Geotechnical Engineer

7.0 References

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Drawings

Geotechnical Investigation

97 Bower Street

759509 Ontario Inc.

SLR Project No.: 243.V24485.00000

October 8, 2025



LEGEND

Borehole Location

Borehole/Monitoring Well Location

Client:	**759509 Ontario Inc.**			Project No.: **243.V24485.00000**	Drawing No.: **1**
Drawn:	**IB**	Approved:	**SM**	Title:	**Borehole / Monitoring Well Location Plan**
Date:	**August, 2025**	Scale:	**As Shown**	Project:	**Geotechnical Investigation - 97 Bower Street, Acton, Ontario**
Original Size:	**Letter**	Rev:	**N/A**	SLR	871 Equestrian Court, Unit 1 Oakville, Ontario, L6L 6L7

Appendix A Borehole Logs

Geotechnical Investigation

97 Bower Street

759509 Ontario Inc.

SLR Project No.: 243.V24485.00000

October 8, 2025

EXPLANATION OF TERMS USED IN THE BOREHOLE LOGS

Sample Type

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Dimension type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Spoon sample
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

Penetration Resistance

Standard Penetration Resistance (SPT), 'N':

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter open sampler for a distance of 300 mm (12 in).

Dynamic Cone Penetration Resistance, N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in) to drive uncased a 50 mm (2 in) diameter 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in).

Textural Classification of Soils

Classification	Particle Size
Boulders	>300 mm
Cobbles	75 mm – 300 mm
Gravel (Gr)	4.75 mm – 75 mm
Sand (Sa)	0.075 mm – 4.75 mm
Silt (Si)	0.002 mm – 0.075 mm
Clay (Cl)	<0.002 mm

Terminology	Proportion
Trace	0 – 10%
Some	10 – 20%
Adjective (e.g. silty or sandy)	20 – 35%
And (e.g. sand and gravel)	> 35 %

Soil Description

a) Cohesive Soils

Consistency	Undrained Shear Strength (kPa)	SPT 'N' Value
Very Soft	< 12	0 – 2
Soft	12 – 25	2 – 4
Firm	25 – 50	4 – 8
Stiff	50 – 100	8 – 15
Very Stiff	100 – 200	15 – 30
Hard	> 200	> 30

b) Cohesionless Soils

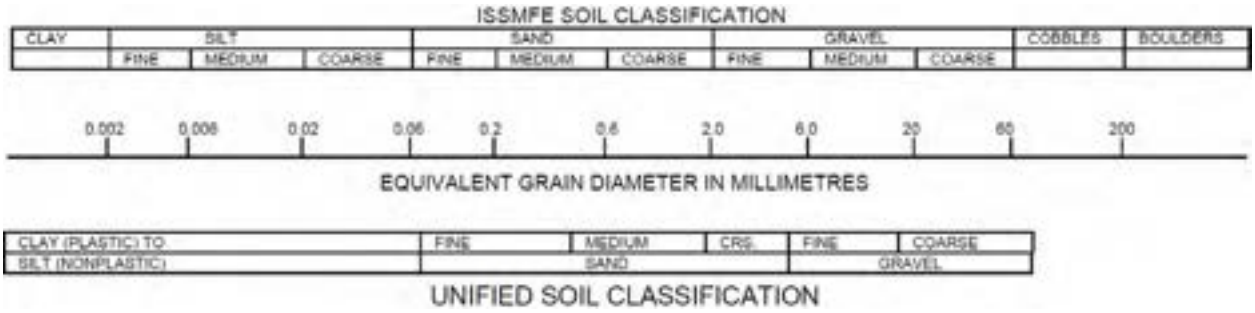
Density Index (Relative Density)	Undrained Shear Strength (kPa)	SPT 'N' Value
Very Loose	N/A	< 4
Loose	N/A	4 – 10
Compact	N/A	10 – 30
Dense	N/A	30 – 50
Very Dense	N/A	> 50

Soil Tests

w	Water content
w_p	Plastic limit
w_l	Liquid limit
C	Consolidation (oedometer) test
CID	Consolidated isotropically drained triaxial test
CIU	Consolidated isotropically undrained triaxial test with porewater pressure measurement
D_R	Relative density (Specific gravity, Gs)
DS	Direct shear test
ENV	Environmental / chemical analysis
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified proctor compaction test
SPC	Standard proctor compaction test
OC	Organic content test
V	Field vane (LV – laboratory vane test)
γ	Unit weight

NOTES ON SAMPLE DESCRIPTIONS

1. All sample descriptions included in this report generally follow the Unified Soil Classification system. Laboratory grain size analyses provided by SLR also follow the same system. Different classification systems may be used by others, such as the system by the International Society for Soil Mechanics and Foundation Engineering (ISSMFE). Please note that with the exception of samples where Gradation and / or Atterberg Limits testing have been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between classification systems.



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

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and / or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated on the borehole logs. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone, caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

LOG OF BOREHOLE BH24-01

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

PROJECT LOCATION: Acton, Ontario

DATUM: Geodetic

BH LOCATION: See Borehole Location Plan N 4831665.53 E 577416.54

Method: Soild Stem Augers

Diameter: 150 mm

Date: May 1, 2024

REF. NO.: 2402401

ENCL NO.: 1

(m) ELEV / DEPTH	CLASSIFICATION	NUMBER	TYPE	BLOWS / 0.3 m "N"	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.6	Ground Surface					
350.6 / 0.0	**TOPSOIL:** 80mm				Concrete	
0.1	**FILL:** silty sand to sandy silt, trace clay, trace gravel, trace organics, brown, moist, very loose to compact.	1	SS	4	Sand	
					350	
		2	SS	15	Bentonite	
					Sand	
					W. L. 349.2 m May 9, 2024	
2	wet	3	SS	11		
348.3						
2.3	**SANDY SILT:** trace clay, trace gravel, dilatant, brown, wet to saturated, very loose to loose.	4	SS	5	348	
3					Screen	
	contains sand and silt layers	5	SS	3		4 45 48 3
					347	
4		6	SS	4		
					346	
5		7	SS	9	Bentonite	
345.3						
5.3	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole. Water Level Readings: Date W. L. Depth (mBGS) 2024-05-09 1.42					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

SLR

LOG OF BOREHOLE BH24-02

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Solid Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 150 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: May 1, 2024

ENCL NO.: 2

BH LOCATION: See Borehole Location Plan N 4831636.74 E 577437.7

ELEV / DEPTH (m)	CLASSIFICATION	NUMBER	TYPE	BLOWS / 0.3 m "N"	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.4	Ground Surface					
350.3 / 0.1	**TOPSOIL:** 70mm				Concrete	
	FILL: silty sand to sandy silt, trace clay, trace gravel, trace organics, brown, moist to saturated, loose to very loose.	1	SS	7	Sand	
					350	
		2	SS	6	Bentonite	
	wet below 0.9m				W. L. 349.3 m May 9, 2024	
		3	SS	5		4 77 15 4
		4	SS	3	348	
347.4 / 3.1	**SANDY SILT:** trace clay, trace gravel, dilatant, brown, saturated, loose.				Screen	
		5	SS	7	347	
		6	SS	8		
					346	
		7	SS	7	Bentonite	
345.1 / 5.3	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole.					

Water Level Readings:

Date	W. L. Depth (mBGS)
2024-05-09	1.15

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ $^{\varepsilon}$=3% Strain at Failure

I:\SK_PROJECTS\2310_CONTRACT DATE_FORMAT_HIGH_LOGS.GLB
PLOTTED: 2024-05-09 LOG: 2402401 - LOT POWER BY - BH_LOGS_2024-0809.GPJ 4.3.0.25

LOG OF BOREHOLE BH24-03

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Solid Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 150 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: May 1, 2024

ENCL NO.: 3

BH LOCATION: See Borehole Location Plan N 4831613.98 E 577420.37

(m) ELEV / DEPTH	CLASSIFICATION	NUMBER	TYPE	BLOWS / 0.3 m "N"	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.6	Ground Surface										
350.6 / 0.0	**TOPSOIL:** 80mm					Concrete					
350.5 / 0.1	**FILL:** silty sand to sandy silt, trace clay, trace gravel, trace organics, brown, moist to wet, very loose to compact.	1	SS	4		Sand					
						350					
		2	SS	3		Bentonite					
						Sand					
		3	SS	9		W. L. 349.2 m May 9, 2024					
		4	SS	14		348					
347.5						Screen					
3.1	**FILL:** sandy gravel, trace clay, trace silt, trace cobbles, brown, wet, loose.	5	SS	10		347					
		6	SS	10							15 25 55 5
345.7						346					
4.9	**SANDY SILT:** trace clay, brown, wet to saturated, loose.	7	SS	7		Bentonite					
345.2											
5.3	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole. Water Level Readings: Date W. L. Depth (mBGS) 2024-05-09 1.37										

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES $+^{3}$, $\times^{3}$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

I:\SK_PROJECTS\2024_CONTRACT\24.TE.XXXX.1 HIGH\LOGS\GLB PALMER_ROA - 6BHs - LOGS - 2402401 - 97 BOWER ST - BH LOGS - 2024-09-03.GPJ - 4.30.25

LOG OF BOREHOLE BH24-04

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Solid Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 150 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: Apr 30, 2024

ENCL NO.: 4

BH LOCATION: See Borehole Location Plan N 4831626.36 E 577391.17

(m) ELEV / DEPTH	CLASSIFICATION	STRATA PLOT	NUMBER	TYPE	BLOWS / 0.3 m "N"	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.1	Ground Surface											
350.0 / 0.1	**TOPSOIL:** 90mm						Concrete					
	FILL: silty sand to sandy silt, trace clay, trace gravel, trace organics, brown, moist, loose to compact.		1	SS	3		Sand					
	wet below 0.8m		2	SS	11		Bentonite W. L. 349.2 m May 9, 2024					
			3	SS	9		Sand					
			4	SS	9		348					0 35 60 5
347.0 / 3.1	**SANDY SILT:** trace clay, dilatant, contains silt seams, brown, wet, loose.		5	SS	9		Screen					
			6	SS	6		346					
			7	SS	6		Bentonite					
344.8 / 5.3	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole. Water Level Readings: Date W. L. Depth (mBGS) 2024-05-09 0.9											

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES + 3, × 3: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

0 SOIL JOCAVES 0101 CONTRACT DATE FORMAT HIGH LOGO.GLB
PLMER ROW - BRUK LOG 2402401 - ILE POWER.GLB - BH LOGS 2024.08.03.GPJ 4.33.25

SLR

LOG OF BOREHOLE BH24-05

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Solid Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 150 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: Apr 30, 2024

ENCL NO.: 5

BH LOCATION: See Borehole Location Plan N 4831596.48 E 577429.53

ELEV / DEPTH (m)	CLASSIFICATION	NUMBER	TYPE	BLOWS / 0.3 m "N"	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.5	Ground Surface					
350.4 / 0.1	**TOPSOIL:** 70mm					
	FILL : sandy silt to silty sand, trace clay, trace gravel, trace organics, greyish brown, moist to wet, loose to compact.	1	SS	5		
		2	SS	8		
	contains rock debris below 1.8m	3	SS	15		
		4	SS	22		
347.4 / 3.1	**FILL:** sandy gravel, trace clay, trace silt, brown, moist to wet, compact.	5	SS	21		
		6	SS	13		23 37 31 9
345.9 / 4.6	**FILL:** sandy silt, trace clay, trace gravel, contains plastic pieces, dilatant, brown, moist to wet, loose.	7	SS	9		
345.1 / 5.3	**END OF BOREHOLE**					

1) 50mm diameter monitoring well was installed upon completion in the borehole.

Water Level Readings:

Date	W. L. Depth (mBGS)
2024-05-09	1.36

Concrete

Sand

350

Bentonite

Sand

W. L. 349.1 m
May 9, 2024

348

Screen

347

346

Bentonite

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%)
10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES

+3, ×3: Numbers refer to Sensitivity

○ ε=3% Strain at Failure

I:\SK_PROJECTS\2313_CONTRACT DATE_FORMAT_HIGH_LOGS.GLB
PLOTTED: 2024-05-09 | LOG: 2402401 | SITE POWER BF | BH LOGS 2024-05-09.GPJ 4.30.25

SLR

LOG OF BOREHOLE BH24-06

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Solid Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 150 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: May 1, 2024

ENCL NO.: 6

BH LOCATION: See Borehole Location Plan N 4831564.67 E 577473.61

(m) ELEV / DEPTH	CLASSIFICATION	NUMBER	TYPE	BLOWS / 0.3 m "N"	ELEVATION
351.3	Ground Surface				
350.9 / 0.1	**TOPSOIL:** 80mm				Concrete
0.4	**FILL:** silty sand to sand mixed with clayey silt pockets, trace to some gravel, trace clay, trace organics, contains asphalt debris, greyish brown, moist, compact.	1	SS	13	Sand 351
		2	SS	25	Bentonite
	contains concrete pieces	3	SS	10	Sand 350
		4	SS	11	349
348.3 / 3.1	**FILL:** silty sand, trace clay, trace gravel, brown, wet to saturated, loose.	5	SS	10	Screen W. L. 348.1 m May 9, 2024
347.0 / 4.3	**SANDY SILT:** trace clay, contains sand seams, dilatant, brown, saturated, compact to loose.	6	SS	14	347
		7	SS	6	Bentonite
346.0 / 5.3	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole. Water Level Readings: Date W. L. Depth (mBGS) 2024-05-09 3.24				346

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		9 72 15 4

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity; ○ $^{\varepsilon_f=3\%}$ Strain at Failure

LOG OF BOREHOLE BH24-07

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Hollow Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 200mm

REF. NO.: 2402401

DATUM: Geodetic

Date: May 2, 2024

ENCL NO.: 7

BH LOCATION: See Borehole Location Plan N 4831650.26 E 577417.51

ELEV / DEPTH (m)	CLASSIFICATION	NUMBER	TYPE	BLOWS 0.3 m "N"	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.4	Ground Surface										
350.3 / 0.1	**TOPSOIL:** 60mm					Concrete					
	FILL: silty sand, trace gravel, trace organics, brown, moist, moist to wet, loose to compact.	1	SS	7		Sand					
	wet below 0.8m	2	SS	9	▽	350					
		3	SS	10		W. L. 349.2 m May 9, 2024					
347.3		4	SS	14		348					
3.1	**SANDY SILT:** trace clay, brown, wet to saturated, loose to very loose.	5	SS	7		347					
		6	SS	2		346					
344.7						345					
5.7	**SILT:** trace clay, trace sand, dilatant, brown, wet, very loose to very dense.	7	SS	2		Bentonite 344					
	saturated	8	SS	55		343 / 342					Spoon boucing
		9	SS	15		341					

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity; ε=3% Strain at Failure

I:\SK_PROJECTS\2024_CONTRACT DATE_FORMAT_HIGH_LOGO.GLB
PLAXIS 2D\e_BH24_LOGS_2402401_V8.PLAXIS 2D_BH_LOGS_2024-0803.GPJ 4.33r23

LOG OF BOREHOLE BH24-07

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Hollow Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 200mm

REF. NO.: 2402401

DATUM: Geodetic

Date: May 2, 2024

ENCL NO.: 7

BH LOCATION: See Borehole Location Plan N 4831650.26 E 577417.51

ELEV / DEPTH (m)	CLASSIFICATION	NUMBER	TYPE	BLOWS 0.3 m "N"	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
340.2						
10.2	**SANDY SILT TILL:** trace to some clay, trace gravel, contains dilatant silt layers, brown, wet, compact to very dense.				340	
11		10	SS	22		
					339	
12					Sand	
		11	SS	50/ initial 100mm	338	Spoon boucing
13					337	
					Screen	
14	contains sand and silt layers	12	SS	50/ 75mm		6 43 44 7
335.8					336	
14.6	**CLAYEY SILT TILL:** sandy, trace gravel, trace rock fragments, grey, moist to wet, hard.					
15						
335.0		13	SS	50/ initial 100mm	Bentonite	
15.3	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole. Water Level Readings: Date W. L. Depth (mBGS) 2024-05-09 1.2					Spoon boucing

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH NOTES $+^3$, $\times^3$: Numbers refer to Sensitivity ○ ε=3% Strain at Failure

I:\SK_PROJECTS\2024_CONTRACT\24-TE-XXXXA-T HIGH\LOGS\24-B PALMER.PCN - 6/8/24 - LOG: 2402401 - LIB: POWER.GLB - BH LOGS 2024-06-03.GPJ - 4.32b.25

LOG OF BOREHOLE BH24-08

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

PROJECT LOCATION: Acton, Ontario

DATUM: Geodetic

BH LOCATION: See Borehole Location Plan N 4831644.63 E 577455.82

Method: Hollow Stem Augers

Diameter: 200 mm

Date: May 1, 2024

REF. NO.: 2402401

ENCL NO.: 8

(m) ELEV / DEPTH	CLASSIFICATION	STRATA PLOT	NUMBER	TYPE	BLOWS 0.3 m "N"	GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
350.2	Ground Surface											
350.0 / 0.1	**TOPSOIL:** 80mm						Concrete					
	FILL: silty sand, clayey, trace gravel, trace organics, greyish brown, moist, very loose to loose.		1	SS	1		Sand					
1			2	SS	5	▽	W. L. 349.2 m May 9, 2024					
2			3	SS	7		348					
			4	SS	10		Bentonite					
3			5	SS	8		347					
4							346					
345.6							Sand					
4.6	**SILT:** trace sand, trace to some clay, dilatant, brown, wet to saturated, loose to compact.		6	SS	6							0 5 91 4
5							345					
							Screen					
6	wet below 6.1m		7	SS	15		344					
7							343					
							Bentonite					
8			8	SS	11		342					
341.9 / 8.2	**END OF BOREHOLE** 1) 50mm diameter monitoring well was installed upon completion in the borehole.											

Water Level Readings:

Date	W. L. Depth (mBGS)
2024-05-09	0.96

GROUNDWATER ELEVATIONS

Measurement 1st ▽ 2nd ▼ 3rd ▼ 4th ▼

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH24-09

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

PROJECT LOCATION: Acton, Ontario

DATUM: Geodetic

BH LOCATION: See Borehole Location Plan N 4831634.73 E 577402.62

Method: Hollow Stem Augers

Diameter: 200 mm

Date: May 2, 2024

REF. NO.: 2402401

ENCL NO.: 9

(m) ELEV / DEPTH	CLASSIFICATION	NUMBER	TYPE	BLOWS / 0.3 m "N"	ELEVATION
350.6	Ground Surface				
350.6 / 0.0	**TOPSOIL:** 90mm				
0.1	**FILL:** sand, trace clay, trace silt, trace gravel, trace organics, brown, moist, loose to compact.	1	SS	6	350
1	moist to wet	2	SS	10	
	wet below1.5m	3	SS	8	349
2		4	SS	14	348
3		5	SS	22	347
4					
346.0					
4.6	**SANDY SILT:** trace clay, dilatant, brown, saturated, very loose.	6	SS	4	346
5					
344.9					345
5.6	**SILT:** trace to some sand, trace clay, dilatant, brown, wet, very loose.	7	SS	2	
6					344
7					
343.4					
7.2	**SANDY SILT TILL:** trace to some clay, trace gravel, contains dilatant silt seams, brown, moist, dense.	8	SS	32	343
8					
342.3					
8.2	**END OF BOREHOLE**				

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

REMARKS AND GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

Wet spoon below

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH NOTES: +3, ×3: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

LOG OF BOREHOLE BH24-10

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Hollow Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 200 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: Apr 30, 2024

ENCL NO.: 10

BH LOCATION: See Borehole Location Plan N 4831603.9 E 577435.27

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE & Sensitivity × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30)	POCKET PEN. (Cu) (kPa)	NATURAL UNIT WT (kN/m³)	REMARKS AND GRAIN SIZE DISTRIBUTION (%)
(m) ELEV / DEPTH	CLASSIFICATION	STRATA PLOT	NUMBER	TYPE	"N" BLOWS 0.3 m							GR SA SI CL
350.5	Ground Surface											
350.4 / 0.1	**TOPSOIL:** 100mm											
	FILL: sandy silt to silty sand, trace clay, trace gravel, trace organics, brown, moist to wet, very loose.		1	SS	4		350					
			2	SS	2		349					
			3	SS	3							
348.3							348					
2.2	**FILL:** sand and gravel, trace clay, trace silt, trace organics, brown, wet, compact.		4	SS	23							
			5	SS	19		347					
345.9							346					
4.6	**SANDY SILT:** trace clay, trace gravel, dilatant, brown, wet, loose		6	SS	7							
344.9							345					
5.6	**SILT:** trace to some clay, trace sand, dilatant, brown, saturated, very loose to dense.		7	SS	4		344					
			8	SS	31		343					
							342					
			9	SS	11		341					

Continued Next Page

GROUNDWATER ELEVATIONS

Measurement 1st 2nd 3rd 4th

GRAPH NOTES: $+^3$, $\times^3$: Numbers refer to Sensitivity; ○ ε=3% Strain at Failure

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PLANEX_ROW_1994_1.0GO_2402401_1.0 BOWER.GLB - BH LOGS_2024_0809.GPJ 4.3.0.23

LOG OF BOREHOLE BH24-10

PROJECT: Geotechnical Investigation - 97 Bower Street

CLIENT: 759509 Ontario Inc.

Method: Hollow Stem Augers

PROJECT LOCATION: Acton, Ontario

Diameter: 200 mm

REF. NO.: 2402401

DATUM: Geodetic

Date: Apr 30, 2024

ENCL NO.: 10

BH LOCATION: See Borehole Location Plan N 4831603.9 E 577435.27

(m) ELEV / DEPTH	CLASSIFICATION	STRATA PLOT	NUMBER	TYPE	BLOWS 0.3 m "N"	GROUND WATER CONDITIONS	ELEVATION	REMARKS AND GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued							
	SILT: trace to some clay, trace sand, dilatant, brown, saturated, very loose to dense.(Continued)							
11			10	SS	24		340	
338.8								
11.7	**CLAYEY SILT TILL:** sandy, trace gravel, contains rock fragments, grey, wet, hard.						339	
12			11	SS	93/ 125mm		338	Spoon bouncing
337.4								
13.1	**CLAYEY SILT TILL/SHALE COMPLEX:** sandy, trace gravel, contains rock/shale fragments, grey, wet, hard.							Auger grinding
14			12	SS	50/ initial 125mm		337	Spoon bouncing
15							336	
335.1			13	SS	50/ intial 125mm			Spoon bouncing
15.4	**END OF BOREHOLE**							

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

SHEAR STRENGTH (kPa)
○ UNCONFINED + FIELD VANE & Sensitivity
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%)
10 20 30

POCKET PEN. (Cu) (kPa)

NATURAL UNIT WT (kN/m³)

GROUNDWATER ELEVATIONS
Measurement 1st 2nd 3rd 4th

GRAPH NOTES
+3, ×3: Numbers refer to Sensitivity
○ε=3% Strain at Failure

I:\SOIL (EXCAVATED DATA CORRECT DATE FORMAT HIGH LOGS.GLB
PALMER.ZON - 8.8.0.4 - LOG - 2402401 - 97 BOWER.GPJ - BH LOGS - 2024-08-03.GPJ - 4.33-25

Appendix B Geotechnical Laboratory Results

Geotechnical Investigation

97 Bower Street

759509 Ontario Inc.

SLR Project No.: 243.V24485.00000

October 8, 2025

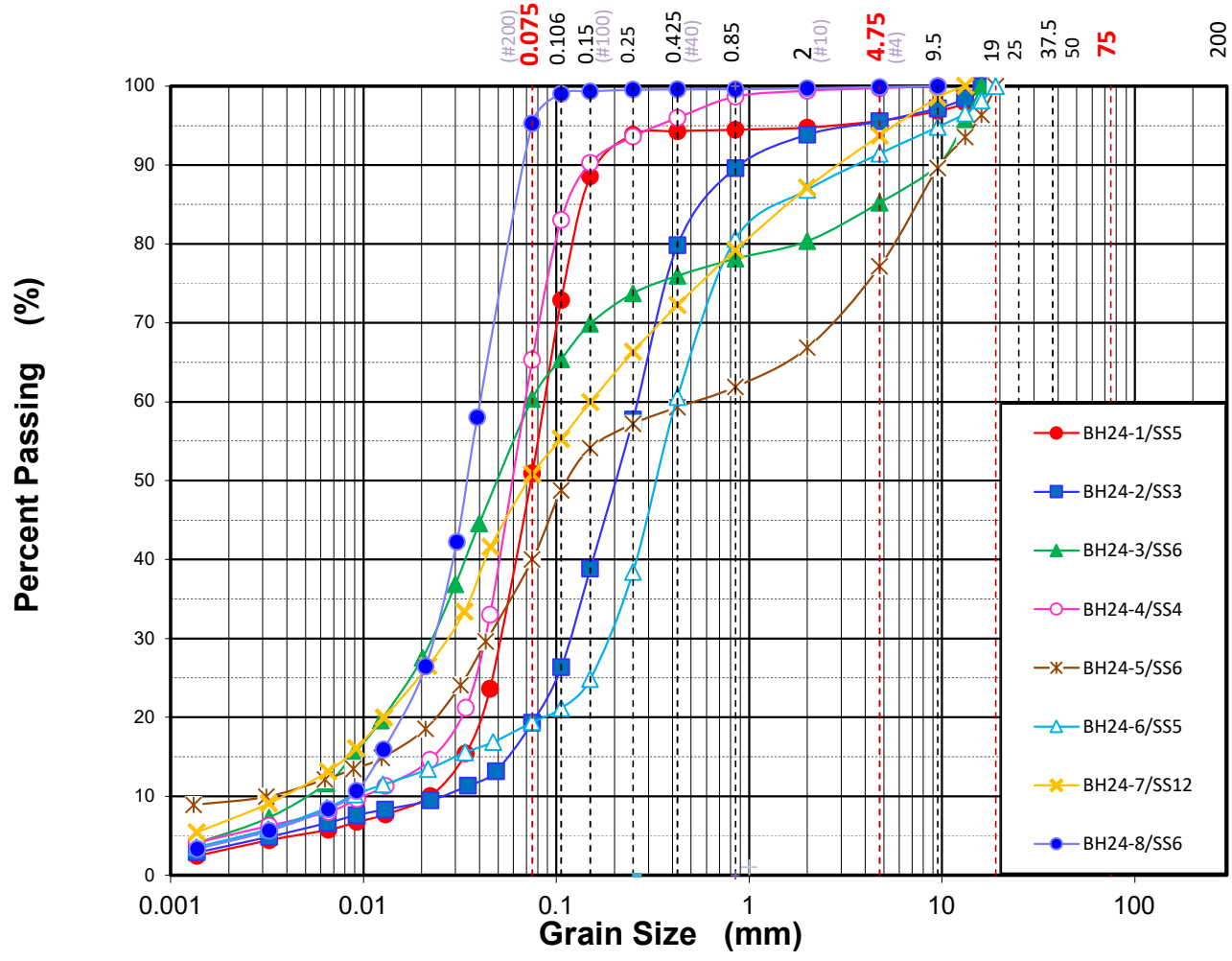
Palmer Environmental Consulting Group Inc.
871 Equestrain Ct, Unit 1
Oakville, ON L6L 6L7

Particle Size Distribution Report
(ASTM D421/422)

Project No.:	2402401	Lab No.:	R24-016
Project Name:	Phase Two ESA	Tested By:	TO
Client:	759509 Ontario Inc.	Checked By:	BW
Location:	97 Bower Street, Acton, Ontario	Date:	5/15/2024

Test Results

Test No.	Sample No.	Clay	Silt	Sand			Gravel		Cobble+	Remarks
				Fine	Medium	Coarse	Fine	Coarse		
1	BH24-1/SS5	3	48	45			4			
2	BH24-2/SS3	4	15	77			4			
3	BH24-3/SS6	5	55	25			15			
4	BH24-4/SS4	5	60	35			0			
5	BH24-5/SS6	9	31	37			23			
6	BH24-6/SS5	4	15	72			9			
7	BH24-7/SS12	7	44	43			6			
8	BH24-8/SS6	4	91	5			0			





Making Sustainability Happen