Geotechnical Investigation

Proposed Car Wash Development
3555 Borrisokane Road
Barrhaven, Ontario

Prepared for:

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LRL File No.: 210691

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1 INTRODUCTION

LRL Associates Ltd. (LRL) was retained by Halo Car Wash Inc. to perform a geotechnical investigation for a proposed car wash development, to be located at 3555 Borrisokane Road, Barrhaven (Nepean), Ontario.

The purpose of the investigation was to identify the subsurface conditions across the site by the completion of a borehole drilling program. Based on the visual and factual information obtained, this report will provide guidelines on the geotechnical engineering aspects of the design of the project, including construction considerations.

This report has been prepared in consideration of the terms and conditions noted above. Should there be any changes in the design features, which may relate to the geotechnical recommendations provided in the report, LRL should be advised in order to review the report recommendations.

2 SITE AND PROJECT DESCRIPTION

The site under investigation is civically located at 3555 Borrisokane Road, Barrhaven, ON. The site fronts Borrisokane Road to the west, Flagstaff Drive to the north, and has a total surface area of about 22,000 m^2. Currently the site is vacant, and at the time of the field investigation the site was snow covered. A stockpile, approximately 3 – 4 m in height is located at the north-west section of the site. Recent aerial photographs indicate the majority of the site has been stripped of vegetation. Excluding the stockpile, the terrain of the site is considered to be relatively flat. The site will be accessible from a future road; Flagstaff Drive. The site location is presented in Figure 1 included in **Appendix A**.

At the time of generating this report, it is understood the development will consist of a drive through car wash complete with access lanes, and vacuum bays. The site will be serviced with municipal services.

3 PROCEDURE

The fieldwork for this investigation was carried out on February 09, 2022. Prior to the fieldwork, the site was cleared for the presence of any underground services and utilities. A total of four (4) boreholes, labelled BH1 through BH4, were drilled across the site, to get a general representation of the site's soil conditions. The approximate locations of the boreholes are shown in Figure 2 included in **Appendix A**.

The boreholes were advanced using a track mount CME 75 drill rig equipped with 200 mm diameter continuous flight hollow stem auger supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. A "two man" crew experienced with geotechnical drilling operated the drill rig and equipment.

Sampling of the overburden materials encountered in the boreholes was carried out at regular depth intervals using a 50.8 mm diameter drive open conventional spoon sampler in conjunction with standard penetration testing (SPT) "N" values. The SPT were conducted following the method **ASTM D1586** and the results of SPT, in terms of the number of blows per 0.3 m of split-spoon sampler penetration after first 0.15 m designated as "N" value.

In-situ field vane shear test using a 125 x 40 mm tapered vane was carried-out in the cohesive soil deposits once the material became very soft based on the “N” values from the blow counts. The undrained shear strength values were calculated following the procedure **ASTM D 2573.**

The boreholes were advanced to depths ranging from 6.71 and 25.83 m below (existing) ground surface (bgs). Upon completion, the boreholes were backfilled using the overburden cuttings.

The fieldwork was supervised throughout by a member of our engineering staff who oversaw the drilling activities, cared for the samples obtained and logged the subsurface conditions encountered within each of the boreholes. All soil samples collected from the boreholes were placed and sealed in plastic bags to prevent moisture loss. The recovered soil samples collected from the boreholes were classified based on visual examination of the materials recovered and the results of the in-situ testing.

Furthermore, all boreholes were located using a Garmin Etrex Legend GPS (Global Positioning System) receiver using NAD 83 datum (North American Datum). LRL’s field personnel determined the existing grade elevations at the borehole locations through a topographic survey carried out using the site bench mark (92.96 m). Ground surface elevations of the boring locations are shown on their respective borehole logs.

4 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

4.1 General

A review of local surficial geology maps provided by the Department of Energy, Mines and Resources Canada suggest that this site is made up of “Champlain Sea Deposits” consisting of blue-grey clay, silt, and silty clay.

The subsurface conditions encountered in the boreholes were classified based on visual and tactile examination of the materials recovered from the boreholes and the results of in-situ laboratory testing. The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil were conducted according to the procedure **ASTM D2487** and judgement, and LRL does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The subsurface soil conditions encountered at the boreholes are given in their respective logs presented in **Appendix B**. A greater explanation of the information presented in the borehole logs can be found in **Appendix C** of this report. These logs indicate the subsurface conditions encountered at a specific test location only. Boundaries between zones on the logs are often not distinct, but are rather transitional and have been interpreted as such.

4.2 Fill

At the surface of all boring locations, a layer of fill material was encountered and extended to depths of 1.45 and 2.45 m bgs. This materially was comprised of a heterogeneous mixture of silt, sand and clay, with some gravel. SPTs were carried out in the fill material and the “N” values were found ranging between 8 and 61. It shall be noted the high blow counts from the surface spoon sample indicate the material was frozen, and not the compactness. The natural moisture content was found to range between 12 and 22%.

4.3 Sandy Clay

Beneath the fill in BH1, a layer of sandy clay was encountered and extended to 2.97 m bgs. The material can be described as having some silt, grey, and moist. The “N” values were found to be 15 and 2, indicating the material is stiff to soft. The natural moisture contents were found to be 32 and 35%.

4.4 Silty Sand

Beneath the fill in BH3, a layer of silty sand was encountered and extended to 2.21 m bgs. The material can be described as having some silt, grey, and moist. The “N” value was found to be 4, indicating the material is loose. The natural moisture contents were found to be 16 and 21%.

4.5 Sandy Silt

Underlying the fill in BH4, a layer of sandy silt was encountered and extended to 2.21 m bgs. The material can be described as having some silt, some gravel sized stone, some clay, brownish grey, and moist. The “N” value was found to be 5, indicating the material is loose.

4.6 Clayey Silt

Underlying the silty sand in BH3, a layer of clayey silt was encountered and extended to an inferred depth of 16.77 m bgs. The material can be described as having some sand, brownish grey, and wet. The “N” values were found ranging from 3 to weight of hammer (WH) indicating the material is soft to very soft. The natural moisture content was found to be 30%.

The undrained shear strength values of this layer ranged between 30 and 40 kPa.

4.7 Silty Clay

Underlying the sandy clay in BH1, the fill in BH2, and the sandy silt in BH4 a layer of silty clay was encountered and extended to a depths ranging from 6.71 (end of exploration) and an inferred depth of 20.73 m bgs. The material can be described as grey in colour, and wet. The “N” values were found ranging from 2 to weight of hammer (WH) indicating the material is very soft. The natural moisture contents were found to be 25 and 46%.

The undrained shear strength values of this layer ranged between 23 and 48 kPa.

4.8 Inferred Glacial Till

Beneath the silty clay in BH1, and the clayey silt in BH3, a deposit of glacial till was encountered and advanced until refusal (>100 blows for 300 mm of penetration) at depths of 25.83 and 21.52 m bgs respectively.

4.9 Laboratory Analysis

Select soil samples were collected for laboratory gradation analyses. The gradation analyses comprised of sieve and hydrometer were conducted following the procedure **ASTM D422.** Details of laboratory analyses are reflected in **Table 1**.

Table 1: Gradation Analysis Summary

Sample Location	Depth (m)	Percent for Each Soil Gradation							Estimated Hydraulic Conductivity K (m/s)
		Gravel		Sand			Silt (%)	Clay (%)	
		Coarse (%)	Fine (%)	Coarse (%)	Medium (%)	Fine (%)			
BH3	6.1 – 6.7	0.0	0.0	0.0	0.0	10.1	56.3	33.6	5 x 10^{-7}
BH4	1.5 – 2.1	6.8	6.1	2.4	5.5	20.0	43.8	15.4	5 x 10^{-6}

Atterberg limits and moisture contents were conducted on a of soil sample from BH1 at depths between 2.3 and 2.9 m. A summary of these values are provided below in **Table 2**.

Table 2: Summary of Atterberg Limits and Water Contents

Sample Location	Parameter					
	Depth (m)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Water Content (%)	USCS Group Symbol
BH1	2.3 – 2.9	25	17	8	32	CL

The laboratory reports can be found in **Appendix D** of this report.

4.10 Groundwater Conditions

Groundwater was carefully monitored during the drilling activities. The soil samples became saturated at depths around 3.0 m indicating the presence of groundwater. It shall be noted no long-term static groundwater monitoring was carried out.

It should be noted that groundwater levels could fluctuate with seasonal weather conditions, (i.e.: rainfall, droughts, spring thawing) and due to construction activities at or near the vicinity of the site.

5 GEOTECHNICAL CONSIDERATIONS

This section of the report provides general geotechnical recommendations for the design aspect of the project based on our interpretation of the information gathered from the boreholes performed at this site and from the project requirements.

This section will detail design parameters for the specific requirements and limitations with regard to allowable foundation bearing pressure and depth, grade raise and size of the footings.

5.1 Foundations

Depending on the required bearing capacity needed to satisfy the structural loading for the car wash building, the proposed building will either be supported by deep foundations (steel driven piles) or shallow foundations (conventional strip and pad footings).

5.1.1 Shallow Foundation

Conventional strip and column footings founded over the undisturbed native material (consisting of a combination of sandy silt, silty sand, and/or silty clay) may be designed using a maximum allowable bearing pressure of **75 kPa** for serviceability limit state **(SLS)** and **110 kPa** for ultimate limit state **(ULS)** factored bearing resistance. The factored ULS value includes the geotechnical resistance factor of 0.5. This bearing capacity limits the allowable grade raise to 1.0 m (above existing grade) and a strip footing maximum width of 2.0 m, and a pad footing maximum width of 4.0 m on any side. This bearing capacity also assumes that the founding depth will not exceed 1.8 m below existing grade.

In-situ field testing may be required to check the strength and stability of the footings subgrade. Any incompetent subgrade areas as identified from in-situ testing must be sub-excavated and backfilled with approved structural fill. Similarly, any soft or wet areas should also be sub-excavated and backfilled with approved structural fill only. Prior to placing any approved structural fill, the subgrade should be inspected and approved by geotechnical engineer or qualified geotechnical personnel. The bearing pressure is contingent on the water level being 0.3 m below the underside footing elevation in order to have a stable and dry subgrade during construction.

Prior to pouring footings concrete the subgrade should be inspected and approved by a geotechnical engineer or a representative of geotechnical engineer

5.1.2 Deep Foundation (Steel Driven Piles)

If a greater bearing capacity is required than what is specified above in Section 5.1.1, consideration shall be given for supporting the foundation on deep foundations. The most common and typically cost-effective deep foundations used in this region are driven steel piles.

The proposed building could be supported on end bearing steel piles driven to refusal within the glacial till and/or bedrock. As most of the overburden soil found on this site is fine grained cohesive soil, it is unlikely that the piles will encounter any significant obstructions during pile installation until refusal is encountered.

Typically, two (2) types of driven steel piles are used within this region. These are as follows:

i. Steel H piles; and

ii. Closed ended, concrete filled, steel pipe piles.

The depth to practical refusal was established to range below about 6.1 to 13.7 m at this site. Generally, the overburden material was the thickest at the west portion of the site and decreased eastward. To minimize the potential for damage to the pile tips during driving, the piles should be provided with a driving shoe as per OPSD standards 3000.100 and 3001.100, for H-pile and steel tube piles, respectively.

Piles driven to refusal generate high ultimate geotechnical capacity, typically equal to the structural capacity of the steel section of the pile. For design example, an HP 310 x 79 with area 9980 mm^2 and yield strength 350 MPa has an un-factored ultimate structural capacity of 3140 kN (assuming structural capacity reduced to 90 percent due to bulking, and lateral loads). The maximum pile capacity for HP 310 x 79 driven to refusal can therefore be considered for **Service Limit State (SLS) 1040 kN** and **Ultimate Limit State (ULS) 1250 kN**. A geotechnical resistance factor 0.4 should be used to the ultimate structural value to obtain the factored ultimate resistance.

Closed ended, concrete filled steel pipe pile of 245 mm diameter can be considered to resist the geotechnical axial resistances as summarized in **Table 3.**

Table 3: Geotechnical Axial Resistance of Steel Pipe Piles

Pile Outside Diameter (mm)	Pipe Wall Thickness (mm)	Geotechnical Axial Resistance	
		Service Limit State (SLS), kN	Ultimate Limit State (ULS), kN
245	9	950	1140
	10	1050	1260
	11	1150	1380

This assumes that the steel has a minimum yield strength of 350 MPa and that the pipe pile is filled with 30 MPa concrete. Pipe piles should be equipped with a base plate having a thickness of at least 20 mm to limit damage to the pile tip during driving.

The piles should be driven no closer than three pile widths/diameters centre to centre.

All of the piles should be driven to refusal. The driving resistance criteria will be highly dependent on the required allowable load and the contractor’s pile driving equipment. Typically, for drop hammer type piling rigs available in Ottawa and surrounding area, a refusal criteria of 20 blows for the last 25 millimetres of penetration would be sufficient to achieve the above allowable loads, assuming that about 35 kilojoules of energy is transferred to the pile per blow.

An allowance should be made in the specifications for this project for re-striking of all the piles at least once to confirm the design set and/or the permanence of refusal and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first re-strike should receive additional re-striking until the design set criteria is met. All re-striking should be performed after 48 hours of the previous set. Furthermore, provisions should be made for dynamic load tests on test piles and for dynamic testing and analysis on selected production piles to verify the driving resistance criteria and pile capacities.

The post construction settlement of elements of the structure, other than the elastic shortening of the piles, should be negligible for end bearing piles driven to refusal over bedrock. For pile foundations, there are no grade raise restrictions.

5.2 Ground Improvements

In lieu of deep foundations, ground improvements could be carried out on this site. In summary, ground improvements methods can consolidate the existing onsite soils, increasing the bearing capacity.

More information regarding this method can be provided via a specialized design-build contractor.

5.3 Structural Fill

For foundations set over undisturbed native soil and where excavation below the underside of the footings is performed in order to reach a suitable founding stratum, consideration should also be given to support the footings on structural fill. The structural fill should be placed over undisturbed native soils in layers not exceeding 300 mm and compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD) within ±2% of its optimum moisture content. In order to allow the spread of load beneath the footings and to prevent undermining during construction, the structural fill should extend minimum 1.0 m beyond the outside edges of the footings and then outward and downward at 1 horizontal to 1 vertical profile (or flatter) over a distance equal to the depth of the structural fill below the footing. Furthermore, the structural fill must be tested to ensure that the specified compaction level is achieved.

5.4 Lateral Earth Pressure

The following equation should be used to estimate the intensity of the lateral earth pressure against any earth retaining structure/foundation walls.

$$P = K(\gamma h + q)$$

Where;

P = Earth pressure at depth h;

K = Appropriate coefficient of earth pressure;

γ = Unit weight of compacted backfill, adjacent to the wall;

h = Depth (below adjacent to the highest grade) at which P is calculated;

q = Intensity of any surcharge distributed uniformly over the backfill surface (usually surcharge from traffic, equipment or soil stockpiled and typically considered 10 kPa).

The coefficient of earth pressure at rest (K_0) should be used in the calculation of the earth pressure on the storm water manhole/basement walls, which are expected to be rather rigid and not to deflect.

The above expression assumes that perimeter drainage system prevents the build-up of any hydrostatic pressure behind the foundation wall.

Table 4 below provides various material types and their respective earth pressure properties.

Table 4: Material and Earth Pressure Properties

Type of Material	Bulk Density (kN/m^3)	Friction Angle (Φ)	Pressure Coefficient		
			At Rest (K_0)	Active (K_A)	Passive (K_P)
Granular A	23.0	34	0.44	0.28	3.53
Granular B Type I	20.0	31	0.49	0.32	3.12
Granular B Type II	23.0	32	0.47	0.31	3.25
Silt to Clay	17.5	19	0.62	0.51	1.97

5.5 Settlement

The estimated total settlement of the shallow foundations, designed using the recommended serviceability limit state capacity value, as well as other recommendations given above, will be less than 25 mm. The differential settlement between adjacent column footings is anticipated to be 15 mm or less.

5.6 Seismic

Based on the results of this geotechnical investigation and in accordance with the Ontario Building Code 2012 (table 4.1.8.4.A.) and Canadian Foundation Engineering Manual (4th edition), the site can be classified as **Class "E"** as per the Site Classification for Seismic Site Response. The underlying soils are not prone to liquefaction.

It should be noted that a greater seismic site response class may be obtained by conducting seismic velocity testing using a multichannel analysis of surface waves (MASW).

The above classifications were recommended based on conventional method exercised for Site Classification for Seismic Site Response and in accordance with the generally accepted geotechnical engineering practice**.**

5.7 Frost Protection

All exterior footings for any heated structure exposed to frost conditions should have a minimum of 1.5 m of earth cover. Footings for any unheated structures, signage or lighting, and where snow will be cleared, 1.8 m of earth cover is required. Alternatively, the required frost protection could be provided using a combination of earth cover and extruded polystyrene insulation. Detailed guidelines for footing insulation frost protection can be provided upon request.

In the event that foundations are to be constructed during winter months, the foundation soils are required to be protected from freezing temperatures using suitable construction techniques. The base of all excavations should be insulated from freezing temperatures immediately upon exposure, until heat can be supplied to the building interior and the footings have sufficient soil cover to prevent freezing of the subgrade soils.

5.8 Foundation Drainage

Permanent perimeter drainage is only required for buildings where basements or whenever any open spaces located below the finish ground are being considered. It is our understanding that no basement construction is included as part of this development and hence no perimeter drainage is required. However, in order to minimize ponding of water adjacent to the foundation walls, roof water should be controlled by a roof drainage system that directs water away from the building to prevent ponding of water adjacent to the foundation wall.

5.9 Foundation Walls Backfill (Shallow Foundations)

To prevent possible foundation frost jacking and lateral loading, the backfill material against any foundation walls, grade beams, isolated walls, or piers should consist of free draining, non-frost susceptible material such as sand or sand and gravel meeting OPSS Granular B Type II or I, or a Select Subgrade Material (SSM).

The foundation wall backfill should be compacted to minimum 95% of its SPMDD using light compaction equipment, where no loads will be set over top. The compaction shall be increased to 98% of its SPMDD under walkways, slabs or paved areas close to the foundation or retaining walls. Backfilling against foundation walls should be carried out on both sides of the wall at the same time where applicable.

5.10 Slab-on-grade Construction

All organic or otherwise deleterious material shall be removed from the proposed building's footprint. The exposed undisturbed native subgrade should then be inspected and approved by a qualified geotechnical personnel.

Any underfloor fill needed to raise the general floor grade shall consist of OPSS Granular B Type II or I, SSM or approved on-site earth borrow, compacted to 98% of its SPMDD. A 200 mm Granular A meeting the **OPSS 1010** shall be placed underneath the slab and compacted to 98% of its SPMDD. Alternatively, if wet condition persists, 200 mm thickness of 19 mm clear stone meeting the **OPSS 1004** requirements shall be used instead of Granular A.

It is also recommended that the area of extensive exterior slab-on-grade (sidewalks, ramp etc.) shall be constructed using Granular A base of thickness 150 mm with incorporating subdrain facilities. The modulus of subgrade reaction (ks) for the design of the slabs set over competent native soil/structural fill is **18 MPa/m.**

In order to further minimize and control cracking, the floor slab shall be provided with wire or fibre mesh reinforcement and construction or control joints. The construction or control joints should be spaced equal distance in both directions and should not exceed 4.5 m. The wire or fibre mesh reinforcement shall be carried out through the joints.

5.11 Corrosion Potential and Cement Type

A soil sample was submitted to Paracel Laboratories Ltd. for chemical testing. The following **Table 5** below summarizes the results.

Table 5: Results of Chemical Analysis

Sample Location	Depth (m)	pH	Sulphate (µg/g)	Chloride (µg/g)	Resistivity (Ohm.cm)
BH4	1.5 – 2.1	7.59	84	30	4,240

Based on the CAN/CSA-A23.1 standards (Concrete Materials and Methods of Concrete Construction), a sulphate concentration of less than 1000 µg/g falls within the negligible category for sulphate attack on buried concrete. The test results from soil samples were below the noted threshold. As such, buried concrete for footings and foundations walls will not require any special additive to resist sulphate attack and the use of normal Portland cement is acceptable.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. Based on the above results, the soil resistivity falls within the corrosive range.

5.12 Other Engineering Considerations

5.12.1 Clay Dykes

As noted above in Section 4.9, the Atterberg Limits results indicate the moisture content is higher than the liquid limit. This indicates that a loss of moisture from the material could result in shrinkage of the soil and subsequent excessive settlements may occur. To help maintain the groundwater level, it is recommended to install clay dykes within service trenches, downstream from each of the manholes/catch basins. These dykes should extend from the base of the service trench to the subgrade level, having minimum width of 1.0 m.

5.12.2 Tree Planting

In addition to clay dykes, any trees planted onsite should have a low demand for water. Trees should be kept at minimum, the anticipated maximum height of the tree away from any buildings/structures.

6 EXCAVATION AND BACKFILLING REQUIREMENTS

6.1 Excavation

It is anticipated that the depth of excavation for the building and any underground services will not be extended below about 1.8 m bgs. Most of the excavation being carried out will be through native fine grained cohesive soils. Excavation must be carried out in accordance with Occupational Health and Safety Act and Regulations for construction Projects.

According to the Ontario's Occupational Health and Safety Act (OHSA), O. Reg. 213/91 and its amendments, the surficial overburden expected to be excavated into at this site can be classified as Type 3. Therefore, shallow temporary excavations can be cut at 1 horizontal to 1 vertical (1H: 1V) for a fully drained excavation starting at the base of the excavation and as per requirements of the OHSA regulations.

Any excavated material stockpiled near an excavation or trench should be stored at a distance equal to or greater than the depth of the excavation/trench and construction equipment, traffic should be limited near open excavation.

6.2 Groundwater Control

Based on the subsurface conditions encountered at this site, minor groundwater seepage or infiltration from the native soils into the shallow temporary excavations during construction is expected. However, it is anticipated that pumping from open sumps should be sufficient to control groundwater inflow. Any groundwater seepage or infiltration entering the excavation should be removed from the excavation by pumping from sumps within the excavations. Surface water runoff into the excavation should be minimized and diverted away from the excavation if possible.

A permit to take water (PTTW) is required from Ministry of Environment and Climate Change (MOECC), Ontario Reg. 387/04, if more than 400,000 litres per day of groundwater will be pumped during a construction period less than 30 days. Registration in the Environmental Activity and Sector Registry (EASR) is required when the takings of ground water and storm water for the purpose of dewatering construction projects range between 50,000 and 400,000 litres per day.

The actual amount of groundwater inflow into open excavations will depend on several factors such as the contractor's schedule, rate of excavation, the size of excavation, depth below the groundwater level, and at the time of year which the excavation is executed. It is expected that pumping rates will be less than 50,000 litres per day. As such, EASR registration is not required for the construction at this site.

6.3 Pipe Bedding Requirements

It is anticipated that the subgrade material for any underground services required as part of this project will be founded over the native silty clay to clayey silt material. Any sub-excavation of disturbed soil should be removed and replaced with a Granular A, Granular B Type II or I or approved equivalent, laid in loose lifts of thickness not exceeding 300 mm and compacted to 95% of its SPMDD. Bedding, thickness of cover material and compaction requirements for any pipes should conform to the manufacturers design requirements and to the detailed installations outlined in the Ontario Provincial Standard Specifications (OPSS) and any applicable standards or requirements. At minimum, a 150 mm thick layer of Granular A shall be used as pipe bedding, at the springline of the pipe, and a 300 mm thick layer above the obvert of the pipe.

If sewers are required to be founded below the groundwater table the native materials may be sensitive to disturbances. Therefore, special precautions should be taken in these areas to stabilize and confine the base of the excavation such as using recompression (thicker bedding) and/or dewatering methods (pumping). In order to properly compact the bedding, the water table should be kept at least 300 mm below the base of the excavation at all time during the installation of any sewers and structures.

As an alternative to Granular A bedding and only where wet conditions are encountered, the use of "clear stone" bedding, such as 19 mm clear stone, **OPSS 1004**, may be considered only in conjunction with a suitable geotextile filter (such as terrafix 270R or approved equivalent). Without proper filtering, there may be entry of fines from native soils and trench backfill into the bedding, which could result in loss of support to the pipes and possible surface settlements. The sub-bedding, bedding and cover materials should be compacted in maximum 300 mm thick lifts to at least 95% of its SPMDD within ±2% of its optimum moisture content using suitable vibratory compaction equipment.

6.4 Trench Backfill

All service trenches should be backfilled using compactable material, free of organics, debris and large cobbles or boulders. Acceptable native materials (if encountered and where possible) should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetrations (i.e. 1.8 m below finished grade) in order to reduce the potential for differential frost heaving between the new excavated trench and the adjacent section of roadway. Where native backfill is used, it should match the native materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type II or I. Any boulders larger than 150 mm in size should not be used as trench backfill.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadway, the trench should be compacted in maximum 300 mm thick lifts to at least 95% of its SPMDD. The specified density may be reduced where the trench backfill is not located within or in close proximity to existing roadways or any other structures.

For trenches carried out in existing paved areas, transitions should be constructed to ensure that proper compaction is achieved between any new pavement structure and the existing pavement structure to minimize potential future differential settlement between the existing and new pavement structure. The transition should start at the subgrade level and extend to the underside of the asphaltic concrete level (if any) at a 1 horizontal to 1 vertical slope. This is especially important where trench boxes are used and where no side slopes are provided to the excavation. Where asphaltic concrete is present, it should be cut back to a minimum of 150 mm from the edge of the excavation to allow for proper compaction between the new and existing pavement structures.

7 REUSE OF ON-SITE SOILS

The existing surficial overburden soils consist mostly of a silts and clays. These materials are considered to be frost susceptible and should not be used as backfill material, except for landscaping purposes where no loads will be applied.

It should be noted that the adequacy of any material for reuse as backfill will depend on its water content at the time of its use and on the weather conditions prevailing prior to and during that time. Therefore, all excavated materials to be reused shall be stockpiled in a manner that will prevent any significant changes in their moisture content, especially during wet conditions. Any excavated materials proposed for reuse should be stockpiled in a manner to promote drying and should be inspected and approved for reuse by a geotechnical engineer.

Any imported material shall conform to OPSS Granular B – Type II or I, SSM, or an approved equivalent.

8 RECOMMENDED PAVEMENT STRUCTURE

It is anticipated that the subgrade soils for the new parking areas/access lanes will consist of sands, silts, clays and possible fill areas. The construction of the parking areas and access lanes will be acceptable over the those subgrade materials once all organic material, or otherwise deleterious material are removed from the subgrade area. Furthermore, the subgrade must be compacted using a suitable heavy duty compacting equipment and approved by a geotechnical engineer prior to placing any granular base material.

The following **Table 6** presents the recommended pavement structures to be constructed over a stable subgrade along the proposed parking areas and access lanes as part of this project.

Table 6: Recommended Pavement Structure

Course	Material	Thickness (mm)	
		Light Duty Parking Area (mm)	Heavy Duty Parking Area (Access Roads, Fire Routes and Trucks) (mm)
Surface	HL3/SP12.5 A/C	50	40
Binder	HL8/SP19.0 A/C	-	50
Base course	Granular A	150	150

Sub base	Granular B Type II	350	450
Total:		550	690

Performance Graded Asphaltic Cement (PGAC) **58-34** is recommended for this project.

The base and subbase granular materials shall conform to **OPSS 1010** material specifications. Any proposed materials shall be tested and approved by a geotechnical engineer prior to delivery to the site and shall be compacted to 98% of its SPMDD. Asphaltic concrete shall conform to **OPSS 1150** and be placed and compacted to at least 93% of the Marshall Density. The mix and its constituents shall be reviewed, tested and approved by a geotechnical engineer prior to delivery to the site.

8.1 Paved Areas & Subgrade Preparation

The access lanes and parking areas shall be stripped of vegetation, debris and other obvious objectionable material. Following the backfilling and satisfactory compaction of any underground service trenches up to the subgrade level, the subgrade shall be shaped, crowned and proof-rolled. A loaded Tandem axle, dual wheel dump truck or approved equivalent heavy duty smooth drum roller shall be used for proof-rolling. Any resulting loose/soft areas should be sub-excavated down to an adequate bearing layer and replaced with approved backfill.

The preparation of subgrade shall be scheduled and carried out in manner so that a protective cover of overlying granular material (if required) is placed as quickly as possible in order to avoid unnecessary circulation by heavy equipment, except on unexcavated or protected surfaces. Frost protection of the surface shall be implemented if works are carried out during the winter season.

The performance of the pavement structure is highly dependent on the subsurface groundwater conditions and maintaining the subgrade and pavement structure in a dry condition. The surface of the pavement should be properly graded to direct runoff water towards suitable drainage features. It is recommended that the lateral extent of the subbase and base layers not be terminated vertically immediately behind the curb/edge of pavement line but be extended beyond the curb.

9 INSPECTION SERVICES

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed site do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design.

All footing areas and any structural fill areas for the proposed structures should be inspected by LRL to ensure that a suitable subgrade has been reached and properly prepared. The placing and compaction of any granular materials beneath the foundations and slab-on-grade should be inspected to ensure that the materials used conform to the grading and compaction specifications.

The subgrade for the pavement areas and underground services should be inspected and approved by geotechnical personnel. In-situ density testing should be carried out on the pavement granular materials, pipe bedding and backfill to ensure the materials meet the specifications for required compaction.

If footings are to be constructed during winter season, the footing subgrade should be protected from freezing temperatures using suitable construction techniques.

10 REPORT CONDITIONS AND LIMITATIONS

It is stressed that the information presented in this report is provided for the guidance of the designers and is intended for this project only. The use of this report as a construction document or its use by a third party beyond the client specifically listed in the report is neither intended nor authorized by LRL Associates Ltd. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible contamination resulting from previous uses or activities at this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report.

The recommendations provided in this report are based on subsurface data obtained at the specific boring locations only. Boundaries between zones presented on the borehole are often not distinct but transitional and were interpreted. Experience indicates that the subsurface soil and groundwater conditions can vary significantly between and beyond the test locations. For this reason, the recommendations given in this report are subject to a field verification of the subsurface soil conditions at the time of construction.

The recommendations are applicable only to the project described in this report. Any changes to the project will require a review by LRL Associates Ltd., to ensure compatibility with the recommendations contained in this project.

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report or if we may be of further services to you, please do not hesitate to contact the undersigned.

Yours truly,
LRL Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
March 9, 2022
B. W. JOHNSON
100510537
PROVINCE OF ONTARIO

Brad Johnson, P.Eng.
Geotechnical Engineer

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APPENDIX A

Site and Borehole Location Plan

LRL

ENGINEERING | INGÉNIERIE

5430 Canotek Road | Ottawa, ON, K1J 9G2
www.lrl.ca | (613) 842-3434

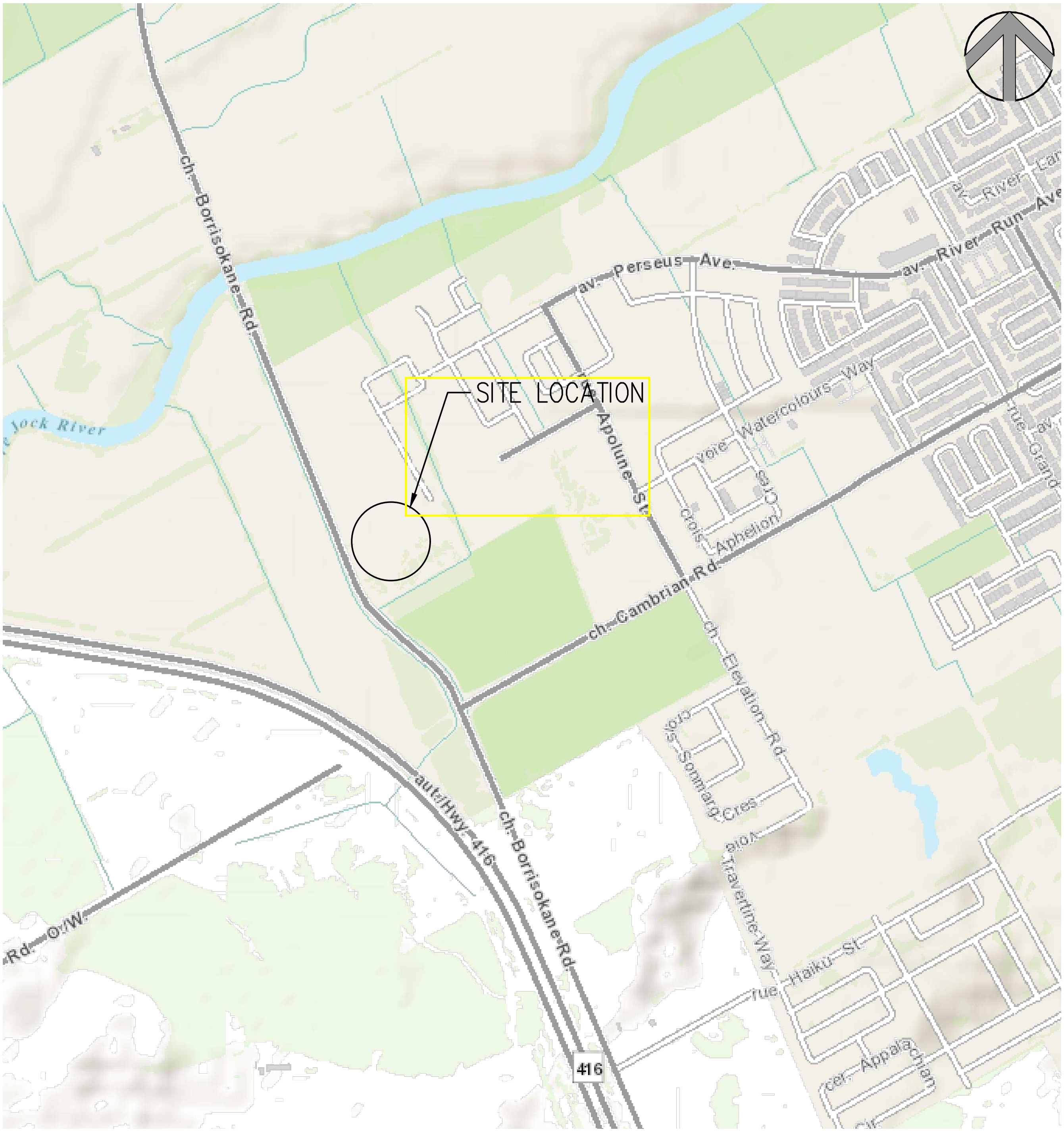
PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED HALO CAR WASH DEVELOPMENT
3555 BORRISOKANE ROAD
OTTAWA, ONTARIO

DRAWING TITLE

SITE LOCATION
SOURCE: GEO-OTTAWA

CLIENT	DATE	PROJECT	
HALO CAR WASH INC.	MARCH 2022	210691	**FIGURE 1**



ENGINEERING | INGÉNIERIE

5430 Canotek Road | Ottawa, ON, K1J 9G2
www.lrl.ca | (613) 842-3434

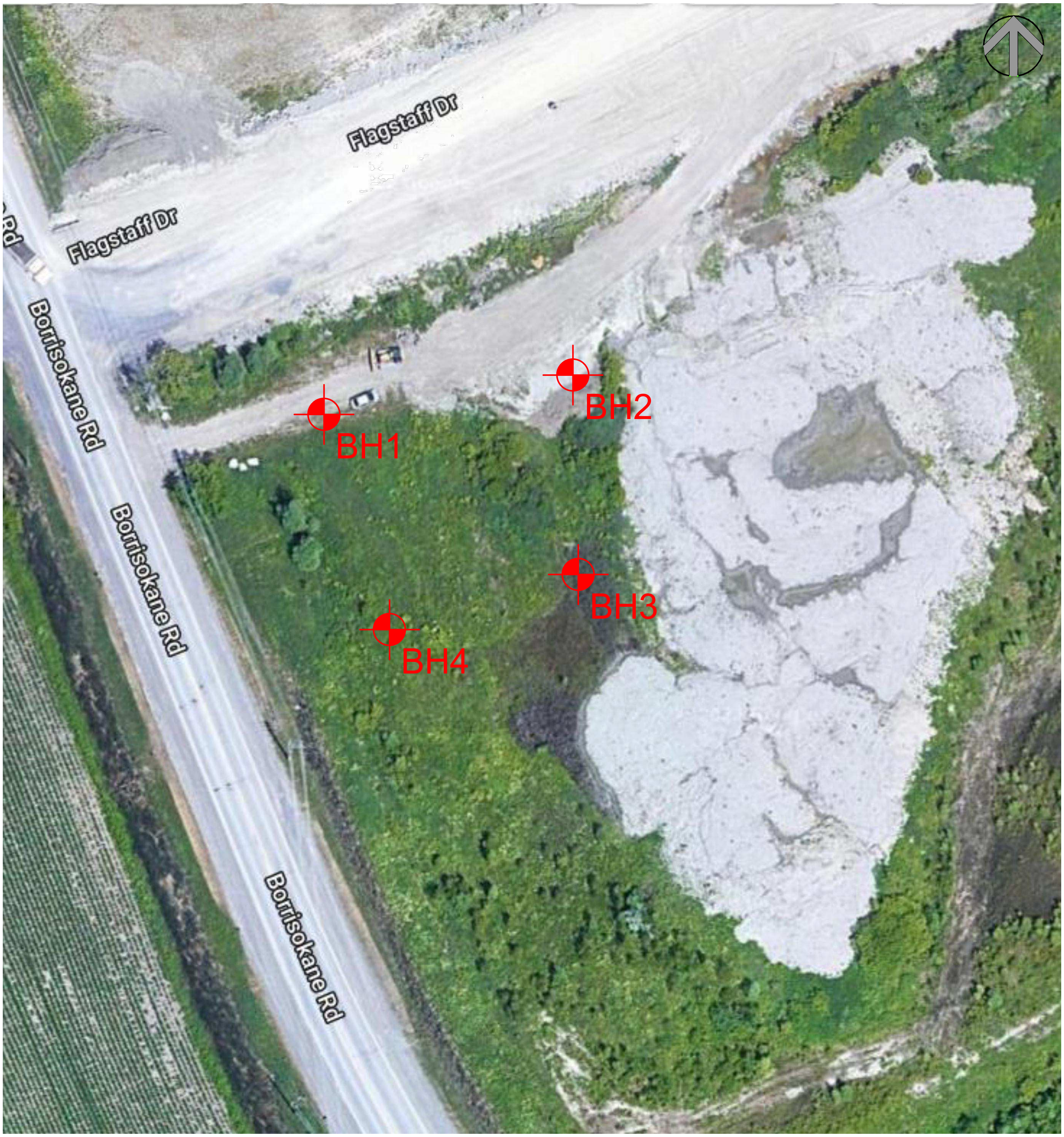
PROJECT

GEOTECHNICAL INVESTIGATION
PROPOSED HALO CAR WASH DEVELOPMENT
3555 BORRISOKANE ROAD
OTTAWA, ONTARIO

DRAWING TITLE

BOREHOLE LOCATION
SOURCE: Imagery 2022 Google, Digital Globe Map Data

CLIENT	DATE	PROJECT	
HALO CAR WASH INC.	MARCH 2022	210691	FIGURE 2



APPENDIX B

Borehole Logs

Borehole Log: BH1

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

Field Personnel: SV

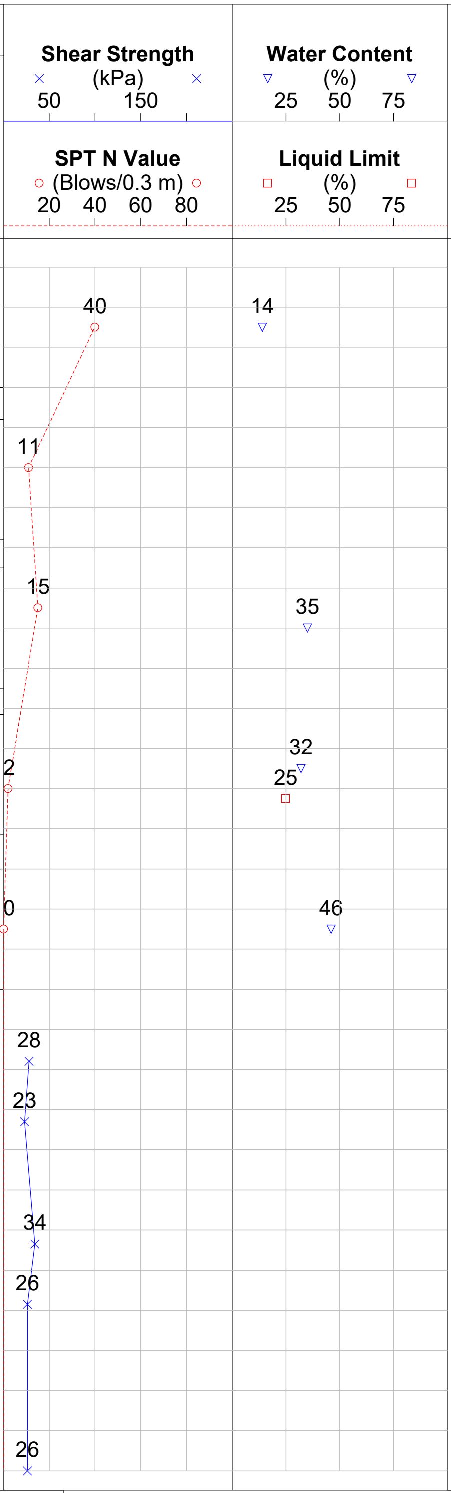
Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE	SAMPLE DATA				
Soil Description	**Elev./Depth (m)**	**Type**	**Sample Number**	**N or RQD**	**Recovery (%)**
Ground Surface	92.59				
FILL sand and gravel, moist, brownish grey, compact. (top 460 mm was frozen)	0.00	⧗	SS1	40	50
		⧗	SS2	11	25
SANDY CLAY some silt, grey, moist, stiff to soft.	91.14 1.45	⧗	SS3	15	50
		⧗	SS4	2	100
SILTY CLAY grey, very soft, wet.	89.62 2.97	⧗	SS5	WH	100

Depth: ft 0–19, m 0–5



Monitoring Well Details

Easting: 440240 m

Northing: 5010691 m

Site Datum: Site Benchmark - 92.96 m.

Groundsurface Elevation: 92.59 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

NOTES:

Borehole Log (continued): BH1

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

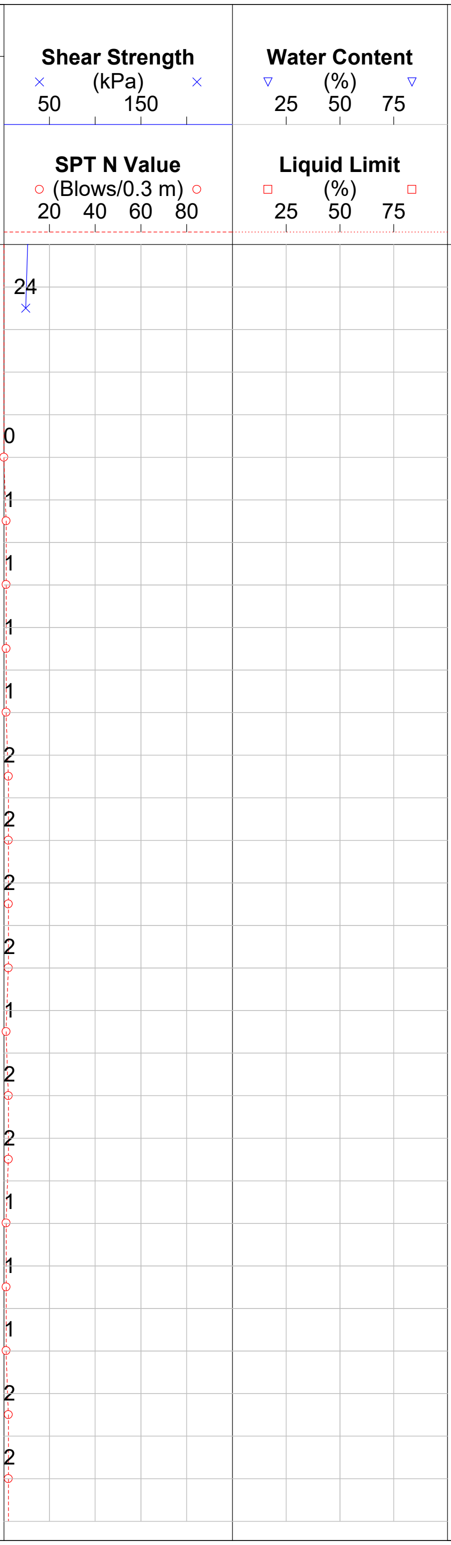
Date: February 9, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

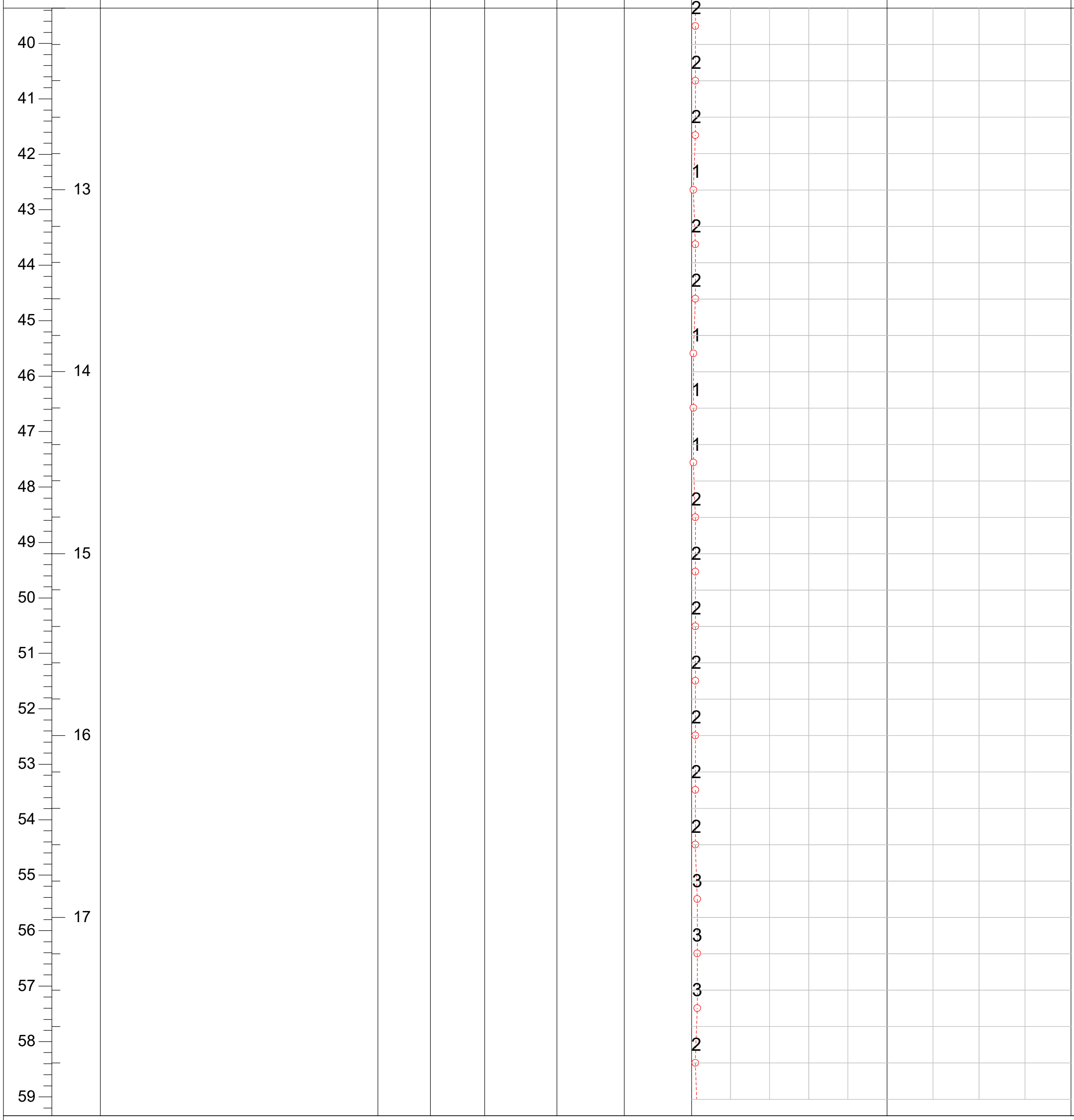
SUBSURFACE PROFILE		SAMPLE DATA							
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength (kPa) 50 150 SPT N Value (Blows/0.3 m) 20 40 60 80	Water Content (%) 25 50 75 Liquid Limit (%) 25 50 75	Monitoring Well Details
20 21 22 23 — 7 24 25 26 — 8 27 28 29 — 9 30 31 32 — 10 33 34 35 36 — 11 37 38 39	Dynamic Cone Penetration Test (DCPT) started at 6.7 m bgs.								

NOTES

Borehole Log (continued): BH1

Project No.: 210691
Project: Proposed Halo Car Wash Development
Client: Halo Car Wash Inc.
Location: 3555 Borrisokane Road, Nepean ON
Date: February 9, 2022
Field Personnel: SV
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Track Mount CME 55
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA							
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength (kPa) × 50 150 × SPT N Value (Blows/0.3 m) ○ 20 40 60 80 ○	Water Content (%) ▽ 25 50 75 ▽ Liquid Limit (%) □ 25 50 75 □	Monitoring Well Details



NOTES

Borehole Log (continued): BH1

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

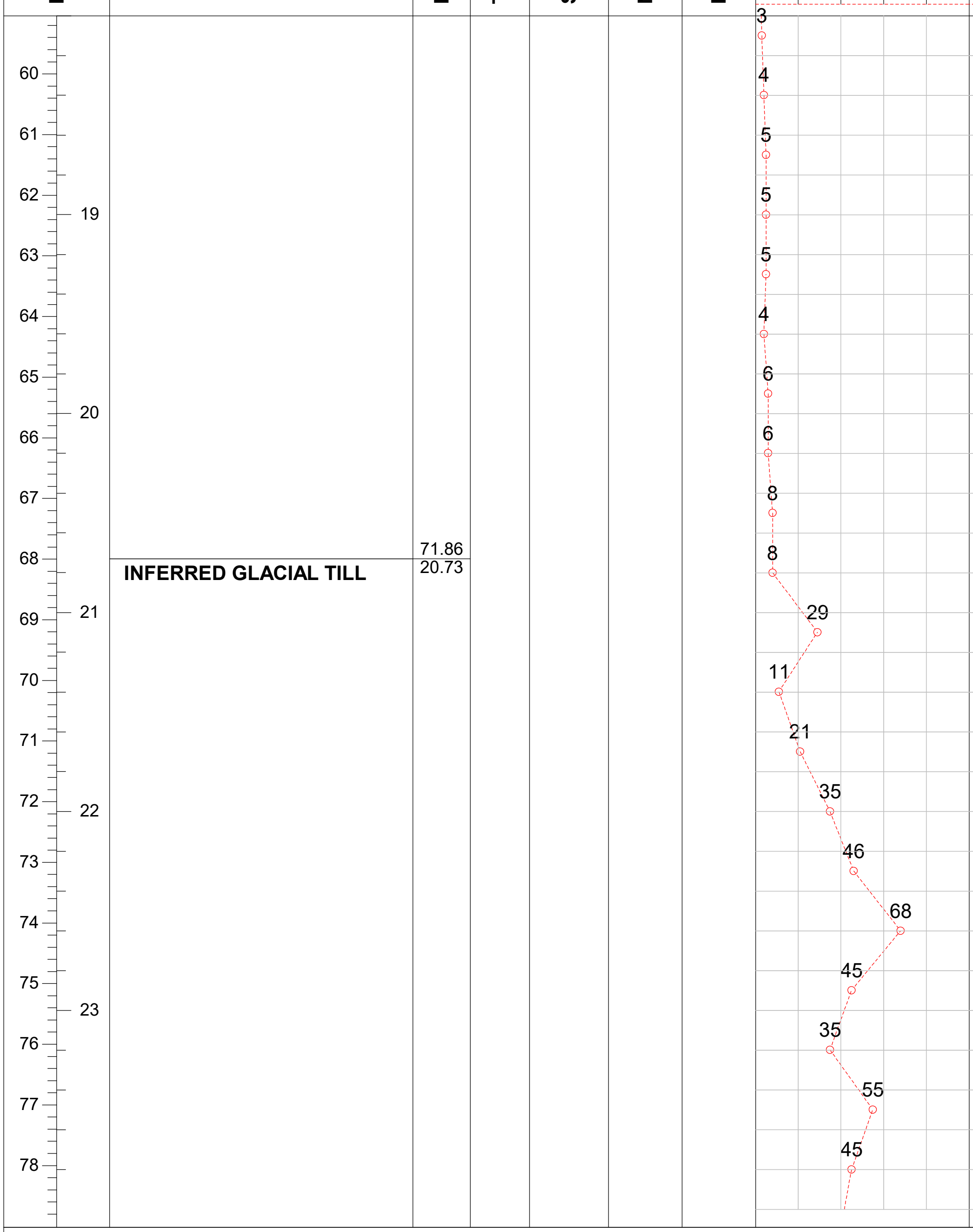
Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA							
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength × (kPa) × 50 150 SPT N Value ○ (Blows/0.3 m) ○ 20 40 60 80	Water Content ▽ (%) ▽ 25 50 75 Liquid Limit □ (%) □ 25 50 75	Monitoring Well Details
	INFERRED GLACIAL TILL	71.86 20.73							



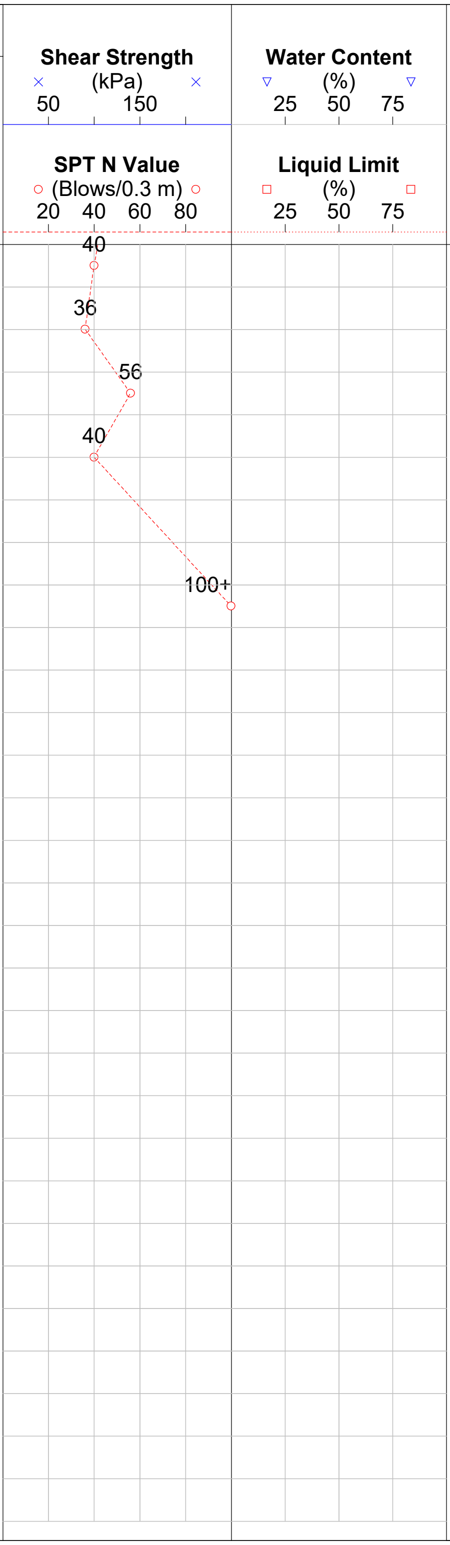
NOTES

Borehole Log (continued): BH1

Project No.: 210691
Project: Proposed Halo Car Wash Development
Client: Halo Car Wash Inc.
Location: 3555 Borrisokane Road, Nepean ON
Date: February 9, 2022
Field Personnel: SV
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Track Mount CME 55
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA							
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength (kPa) / SPT N Value (Blows/0.3 m)	Water Content (%) / Liquid Limit (%)	Monitoring Well Details
79–84 (ft)									
85 (ft) / 26 (m)	End of Borehole	66.76 25.83							

Depth scale: 79, 80, 81, 82, 83, 84, 85, 86, 87, 88, 89, 90, 91, 92, 93, 94, 95, 96, 97, 98 (ft); 25, 26, 27, 28, 29 (m)



NOTES

Borehole Log: BH2

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

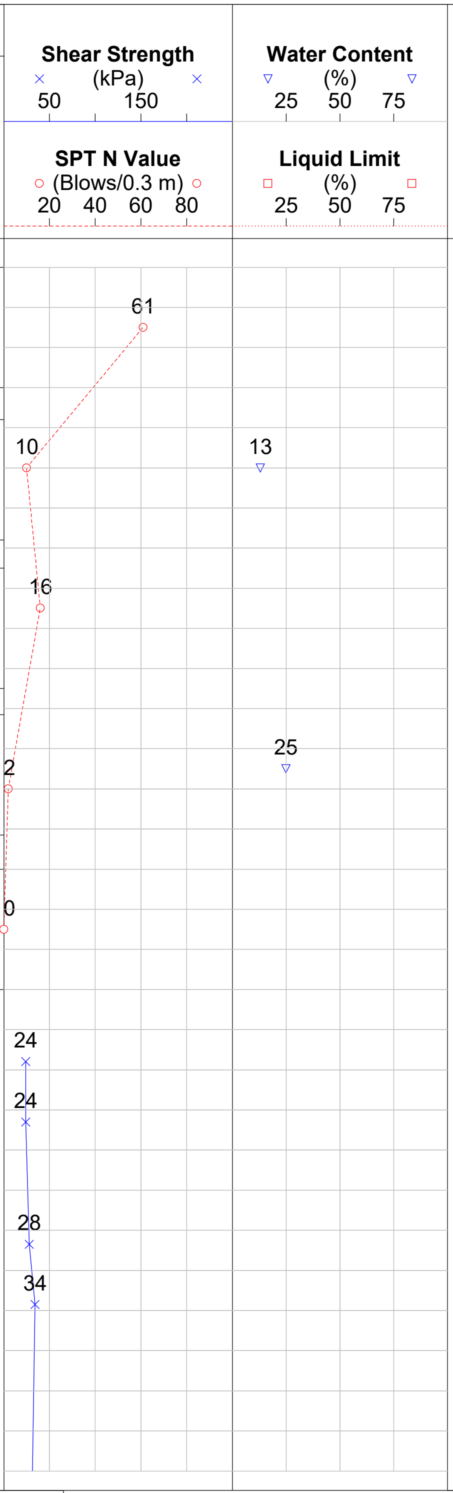
Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE / SAMPLE DATA

Depth (ft / m)	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
0 / 0	Ground Surface	92.80				
0–1.5 ft	**FILL** silt-clay, some gravel, brownish grey, moist, stiff. (top 460 mm was frozen)	0.00	⧗	SS1	61	100
			⧗	SS2	10	17
			⧗	SS3	16	0
	SILTY CLAY trace sand, grey, wet, very soft.	90.59 2.21	⧗	SS4	2	100
			⧗	SS5	0	100

Depth scale: ft 0–19; m 0–5.



Monitoring Well Details

Easting: 440387 m

Northing: 5010744 m

Site Datum: Site Benchmark - 92.96 m.

Groundsurface Elevation: 92.80 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

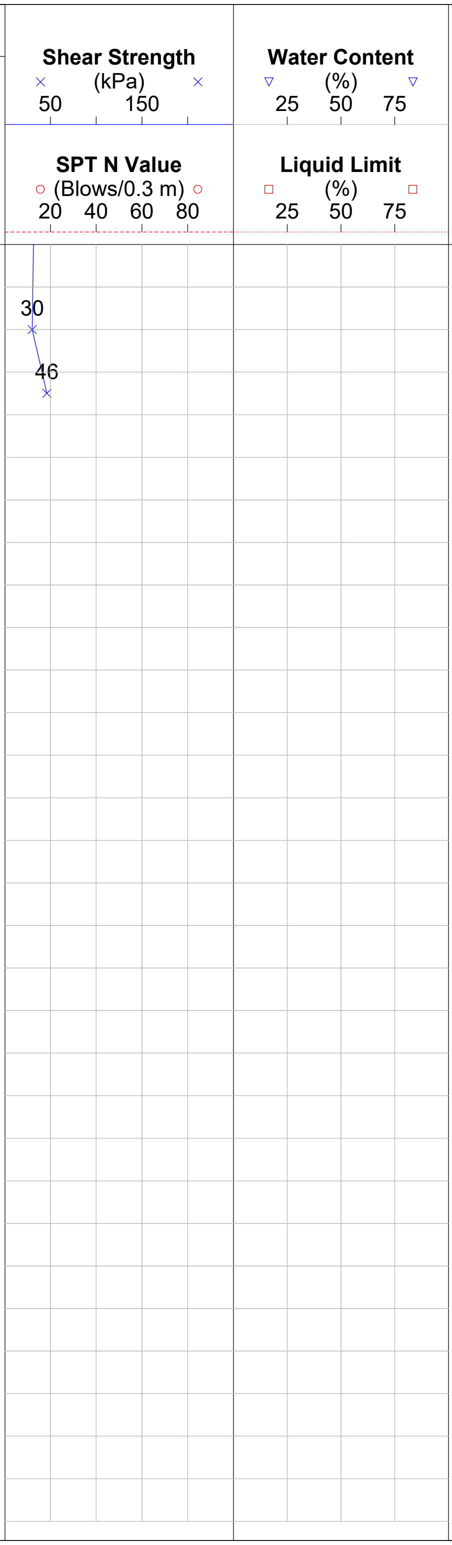
Monitoring Well Diameter: N/A

NOTES:

Borehole Log (continued): BH2

Project No.: 210691
Project: Proposed Halo Car Wash Development
Client: Halo Car Wash Inc.
Location: 3555 Borrisokane Road, Nepean ON
Date: February 9, 2022
Field Personnel: SV
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Track Mount CME 55
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA						Monitoring Well Details
Depth	**Soil Description**	**Elev./Depth (m)**	**Type**	**Sample Number**	**N or RQD**	**Recovery (%)**		
20 21 22								
	End of Borehole	86.09 6.71						
23 — 7 24 25 26 — 8 27 28 29 — 9 30 31 32 — 10 33 34 35 36 — 11 37 38 39								



NOTES

Borehole Log: BH3

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

Field Personnel: SV

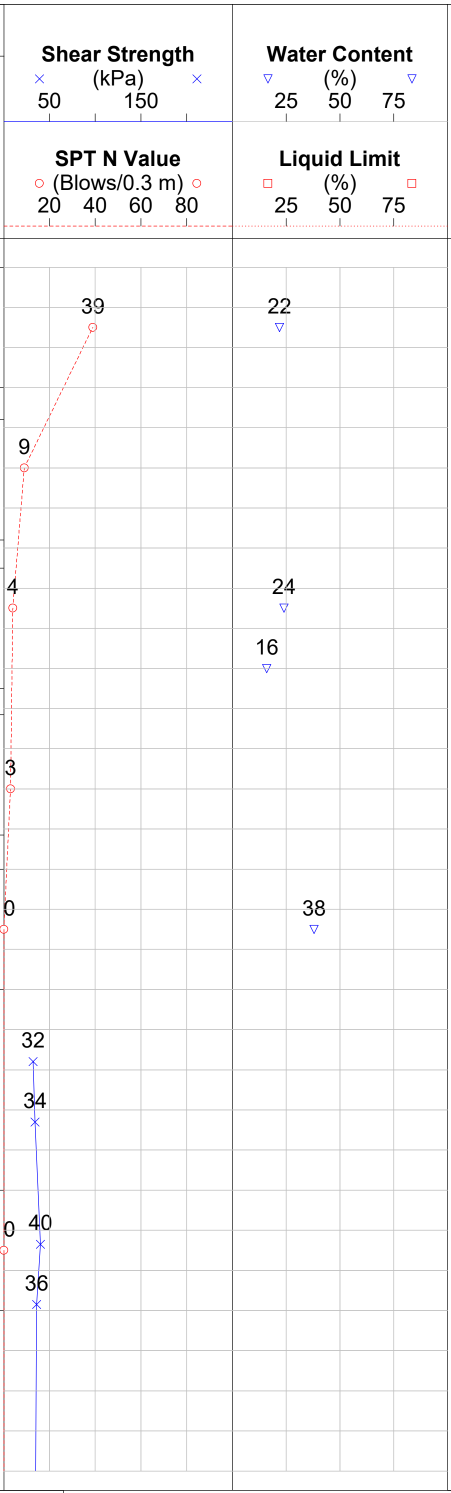
Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE / SAMPLE DATA

Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
	Ground Surface	92.83				
	FILL silt-clay-sand, some gravel, some organic material, brown, moist, loose to compact. (top 460 mm was frozen)	0.00	⧗	SS1	39	50
			⧗	SS2	9	25
	SILTY SAND some clay, brownish grey, moist, loose.	91.38 1.45	⧗	SS3	4	50
	CLAYEY SILT some sand, grey, very soft, wet.	90.62 2.21	⧗	SS4	3	100
			⧗	SS5	WH	100
			⧗	SS6	WH	0



Monitoring Well Details

Easting: 440394 m

Northing: 5010652 m

Site Datum: Site Benchmark - 92.96 m.

Groundsurface Elevation: 92.83 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

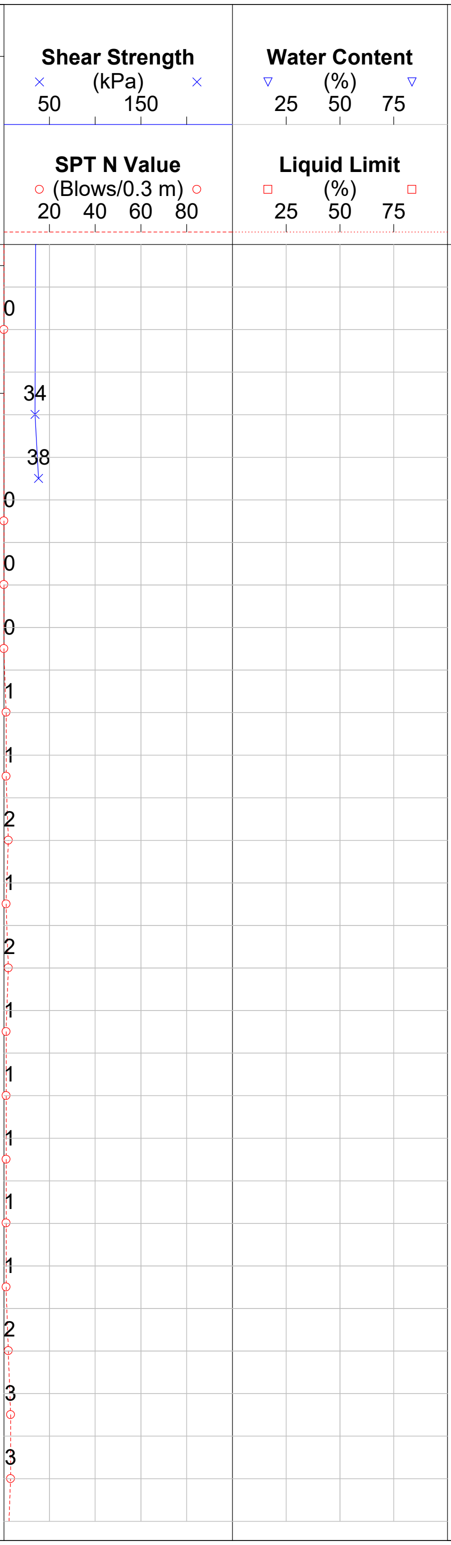
Monitoring Well Diameter: N/A

NOTES:

Borehole Log (continued): BH3

Project No.: 210691
Project: Proposed Halo Car Wash Development
Client: Halo Car Wash Inc.
Location: 3555 Borrisokane Road, Nepean ON
Date: February 9, 2022
Field Personnel: SV
Driller: CCC Geotech and Enviro Drilling
Drilling Equipment: Track Mount CME 55
Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA				
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
20 21			⧗	SS7	WH	100
22 23 – 7 24 25 26 – 8 27 28 29 – 9 30 31 32 33 – 10 34 35 36 – 11 37 38 39	Dynamic Cone Penetration Test (DCPT) started at 7.1 m bgs.					



Monitoring Well Details

NOTES

Borehole Log (continued): BH3

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

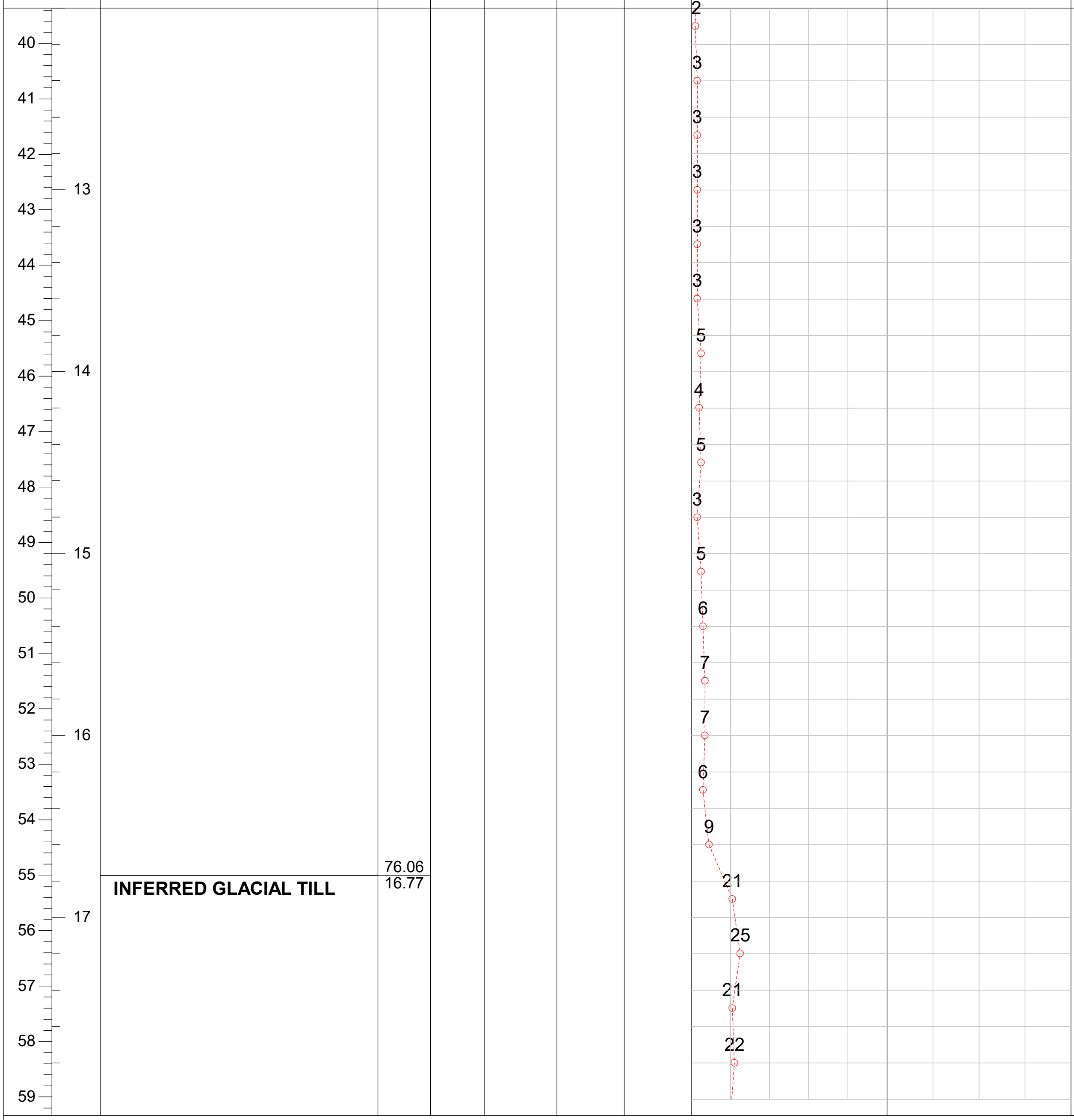
Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA							
Depth	Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)	Shear Strength × (kPa) × 50 150 / SPT N Value ○ (Blows/0.3 m) ○ 20 40 60 80	Water Content ▿ (%) ▿ 25 50 75 / Liquid Limit □ (%) □ 25 50 75	Monitoring Well Details
55	INFERRED GLACIAL TILL	76.06 16.77							



NOTES

Borehole Log (continued): BH3

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

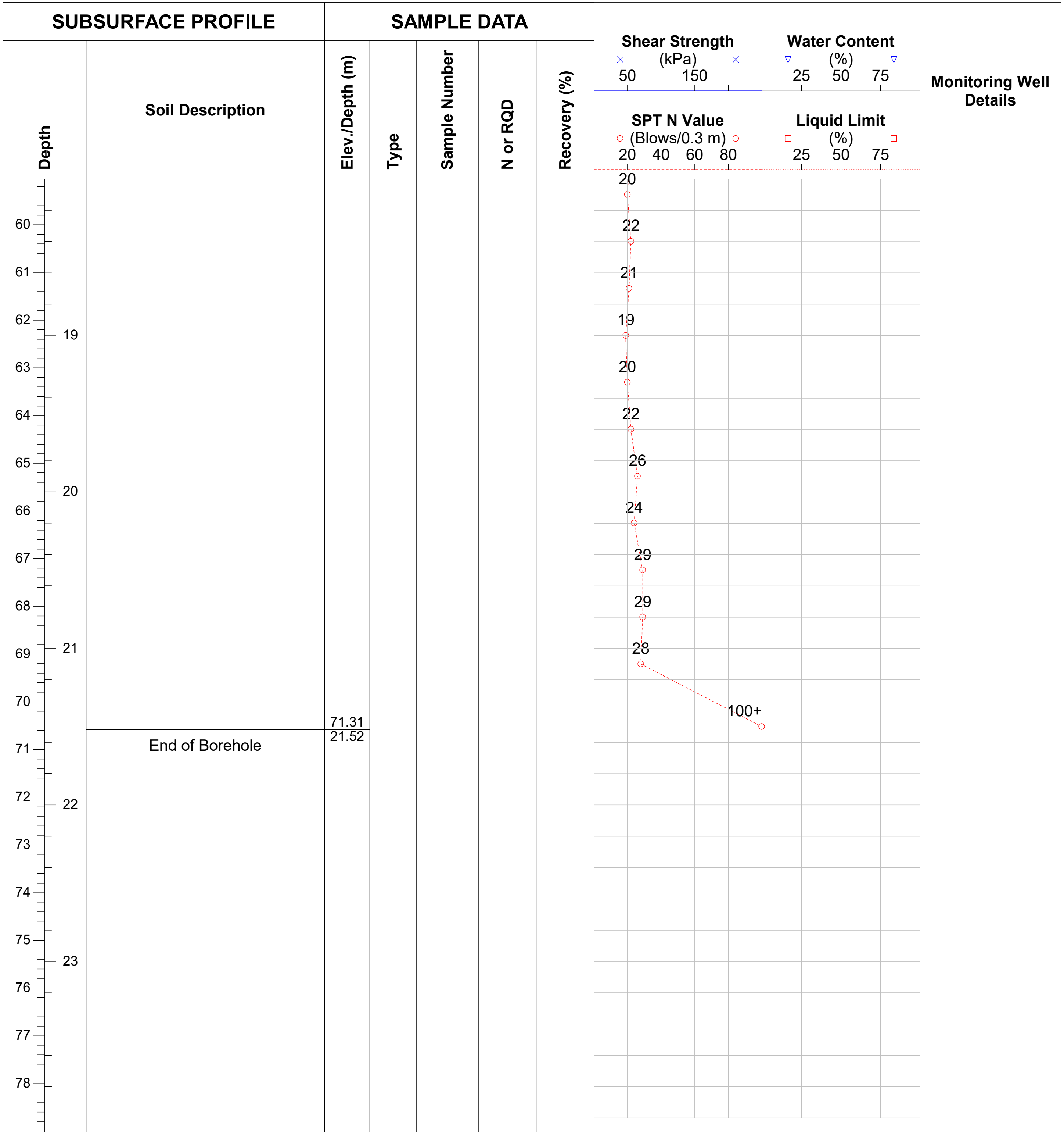
Date: February 9, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger



NOTES

Borehole Log: BH4

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

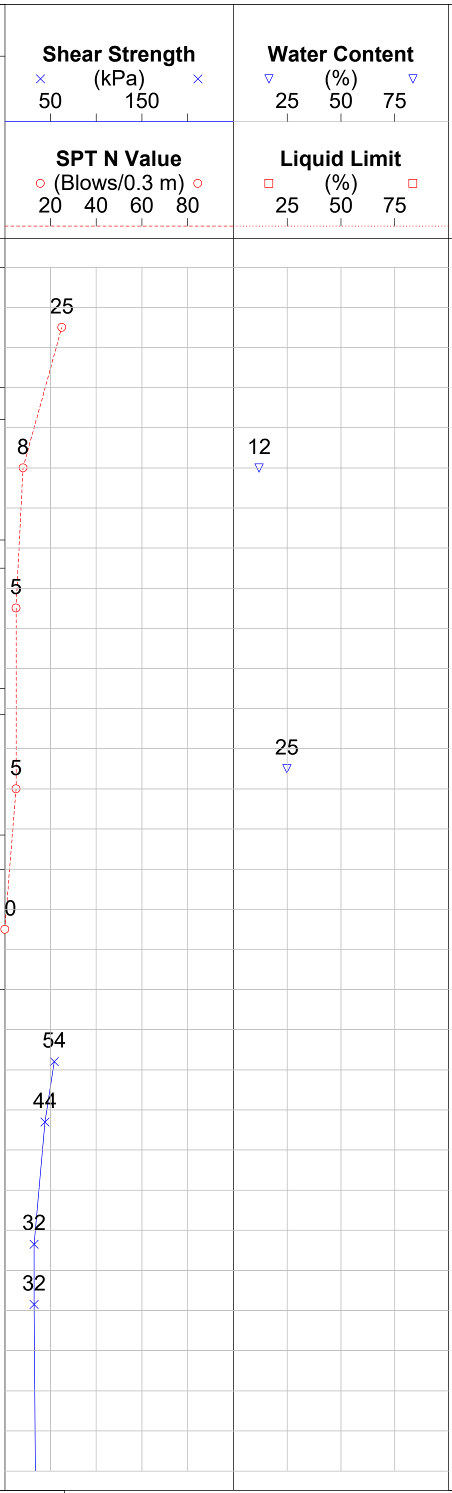
Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE

SAMPLE DATA

Soil Description	Elev./Depth (m)	Type	Sample Number	N or RQD	Recovery (%)
Ground Surface	92.79				
FILL silt-clay, some gravel, brownish grey, moist, stiff. (top 460 mm was frozen)	0.00	⧗	SS1	25	100
		⧗	SS2	8	17
SANDY SILT sand silt, some gravel, some clay, brownish grey, moist, loose.	91.34 1.45	⧗	SS3	5	50
SILTY CLAY trace sand, grey, wet, firm to very soft.	90.58 2.21	⧗	SS4	5	83
		⧗	SS5	0	100



Monitoring Well Details

Easting: 440324 m

Northing: 5010599 m

Site Datum: Site Benchmark - 92.96 m.

Groundsurface Elevation: 92.79 m

Top of Riser Elev.: NA

Hole Diameter: 200 mm

Monitoring Well Diameter: N/A

NOTES:

Borehole Log (continued): BH4

Project No.: 210691

Project: Proposed Halo Car Wash Development

Client: Halo Car Wash Inc.

Location: 3555 Borrisokane Road, Nepean ON

Date: February 9, 2022

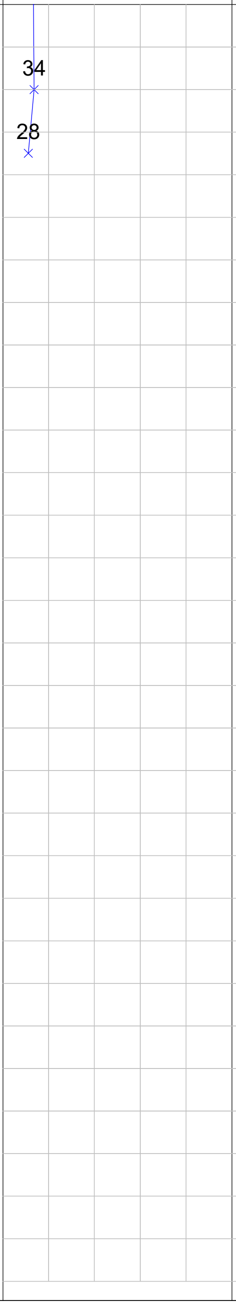
Field Personnel: SV

Driller: CCC Geotech and Enviro Drilling

Drilling Equipment: Track Mount CME 55

Drilling Method: Hollow Stew Auger

SUBSURFACE PROFILE		SAMPLE DATA							
Depth	**Soil Description**	**Elev./Depth (m)**	**Type**	**Sample Number**	**N or RQD**	**Recovery (%)**	**Shear Strength** × (kPa) × 50 150 **SPT N Value** ○ (Blows/0.3 m) ○ 20 40 60 80	**Water Content** ▽ (%) ▽ 25 50 75 **Liquid Limit** □ (%) □ 25 50 75	**Monitoring Well Details**
20 21 22									
23 — 7 24 25 26 — 8 27 28 29 — 9 30 31 32 33 — 10 34 35 36 — 11 37 38 39	End of Borehole	86.08 6.71							



NOTES

APPENDIX C

Symbols and Terms used in Borehole Logs

Symbols and Terms Used on Borehole and Test Pit Logs

1. Soil Description

The soil descriptions presented in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves some judgement and LRL Associates Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice. Boundaries between zones on the logs are often not distinct but transitional and were interpreted.

a. Proportion

The proportion of each constituent part, as defined by the grain size distribution, is denoted by the following terms:

Term	Proportions
"trace"	1% to 10%
"some"	10% to 20%
prefix (i.e. "sandy" silt)	20% to 35%
"and" (i.e. sand "and" gravel)	35% to 50%

b. Compactness and Consistency

The state of compactness of granular soils is defined on the basis of the Standard Penetration Number (N) as per ASTM D-1586. It corresponds to the number of blows required to drive 300 mm of the split spoon sampler using a metal drop hammer that has a weight of 62.5 kg and free fall distance of 760 mm. For a 600 mm long split spoon, the blow counts are recorded for every 150 mm. The "N" value is obtained by adding the number of blows from the 2nd and 3rd count. Technical refusal indicates a number of blows greater than 50.

The consistency of clayey or cohesive soils is based on the shear strength of the soil, as determined by field vane tests and by a visual and tactile assessment of the soil strength.

The state of compactness of granular soils is defined by the following terms:

State of Compactness Granular Soils	Standard Penetration Number "N"	Relative Density (%)
Very loose	0 – 4	<15
Loose	4 – 10	15 – 35
Compact	10 - 30	35 – 65
Dense	30 - 50	65 - 85
Very dense	> 50	> 85

The consistency of cohesive soils is defined by the following terms:

Consistency Cohesive Soils	Undrained Shear Strength (C_u) (kPa)	Standard Penetration Number "N"
Very soft	<12.5	<2
Soft	12.5 - 25	2 - 4
Firm	25 - 50	4 - 8
Stiff	50 - 100	8 - 15
Very stiff	100 - 200	15 - 30
Hard	>200	>30

c. Field Moisture Condition

Description (ASTM D2488)	Criteria
Dry	Absence of moisture, dusty, dry to touch.
Moist	Dump, but not visible water.
Wet	Visible, free water, usually soil is below water table.

2. Sample Data

a. Elevation depth

This is a reference to the geodesic elevation of the soil or to a benchmark of an arbitrary elevation at the location of the borehole or test pit. The depth of geological boundaries is measured from ground surface.

b. Type

Symbol	Type	Letter Code
∫	Auger	AU
⧗	Split Spoon	SS
‖	Shelby Tube	ST
‖	Rock Core	RC

c. Sample Number

Each sample taken from the borehole is numbered in the field as shown in this column.

LETTER CODE (as above) – Sample Number.

d. Recovery (%)

For soil samples this is the percentage of the recovered sample obtained versus the length sampled. In the case of rock, the percentage is the length of rock core recovered compared to the length of the drill run.

3. Rock Description

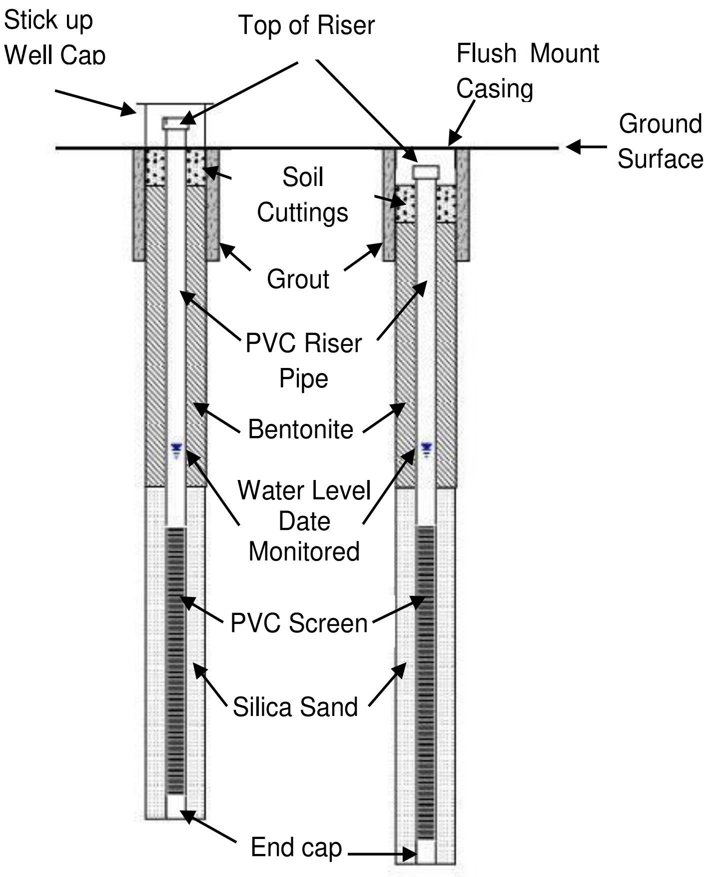
Rock Quality Designation (RQD) is a rough measure of the degree of jointing or fracture in a rock mas. The RQD is calculated as the cumulative length of rock pieces recovered having lengths of 100 mm or more divided by the length of coring. The qualitative description of the bedrock based on RQD is given below.

Rock Quality Designation (RQD) (%)	Description of Rock Quality
0 –25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

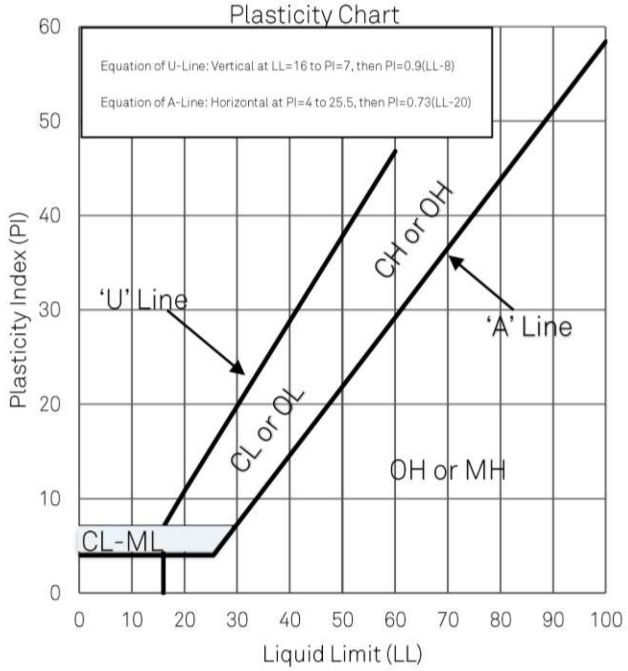
Strength classification of rock is presented below.

Strength Classification	Range of Unconfined Compressive Strength (MPa)
Extremely weak	< 1
Very weak	1 – 5
Weak	5 – 25
Medium strong	25 – 50
Strong	50 – 100
Very strong	100 – 250
Extremely strong	> 250

4. General Monitoring Well Data



5. Classification of Soils for Engineering Purposes (ASTM D2487)
(United Soil Classification System)

Major divisions			Group Symbol	Typical Names	Classification Criteria			
Coarse-grained soils More than 50% retained on No. 200 sieve* (>0.075 mm)	Gravels More than 50% of coarse fraction retained on No. 4 sieve(4.75 mm)	Clean gravels <5% fines	GW	Well-graded gravel	If 15% sand add "with sand" to group name.	Classification on basis of percentage of fines: Less than 5% pass No. 200 sieve - GW, GP, SW, SP More than 12% pass No. 200 sieve - GM, GC, SM, SC 5 to 12% pass No. 200 sieve - Borderline classifications, use of dual symbols	$C_u = \frac{D_{60}}{D_{10}} \geq 4$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravel			Not meeting either C_u or C_c criteria for GW	
		Gravels with >12% fines	GM	Silty gravel			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravel			Atterberg limits on or above "A" line and PI > 7	If fines are organic add "with orgnic fines" to group name
	Sands 50% or more of coarse fraction passes No. 4 sieve(<4.75 mm)	Clean sands <5% fines	SW	Well-graded sand	If 15% gravel add "with gravel to group name		$C_u = \frac{D_{60}}{D_{10}} \geq 6$; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sand			Not meeting either C_u or $C_c$criteria for SW	
		Sands with >12% fines	SM	Silty sand			Atterberg limits below "A" line or PI less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sand			Atterberg limits on or above "A" line and PI > 7	If fines are organic add "with orgnic fines" to group name
Fine-grained soils50% or more passes No. 200 sieve* (<0.075 mm)	Silts and Clays Liquid Limit <50%	Inorganic	ML	Silt	If 15 to 29% coarse-grained, add "with sand" or "with gravel" as appropriate. If > 30% coarse-grained, add "sandy" or "gravelly" as appropriate. Class as organic when oven dried liquid limit is < 75% of undried liquid limit.			
			CL	Lean Clay -low plasticity				
		Organic	OL	Organic clay or silt (Clay plots above 'A' Line)				
	Silts and Clays Liquid Limit >50%	Inorganic	MH	Elastic silt				
			CH	Fat Clay -high plasticity				
		Organic	OH	Organic clay or silt (Clay plots above 'A' Line)				
	Highly Organic Soils		PT	Peat, muck and other highly organic soils				

APPENDIX D

Laboratory Results

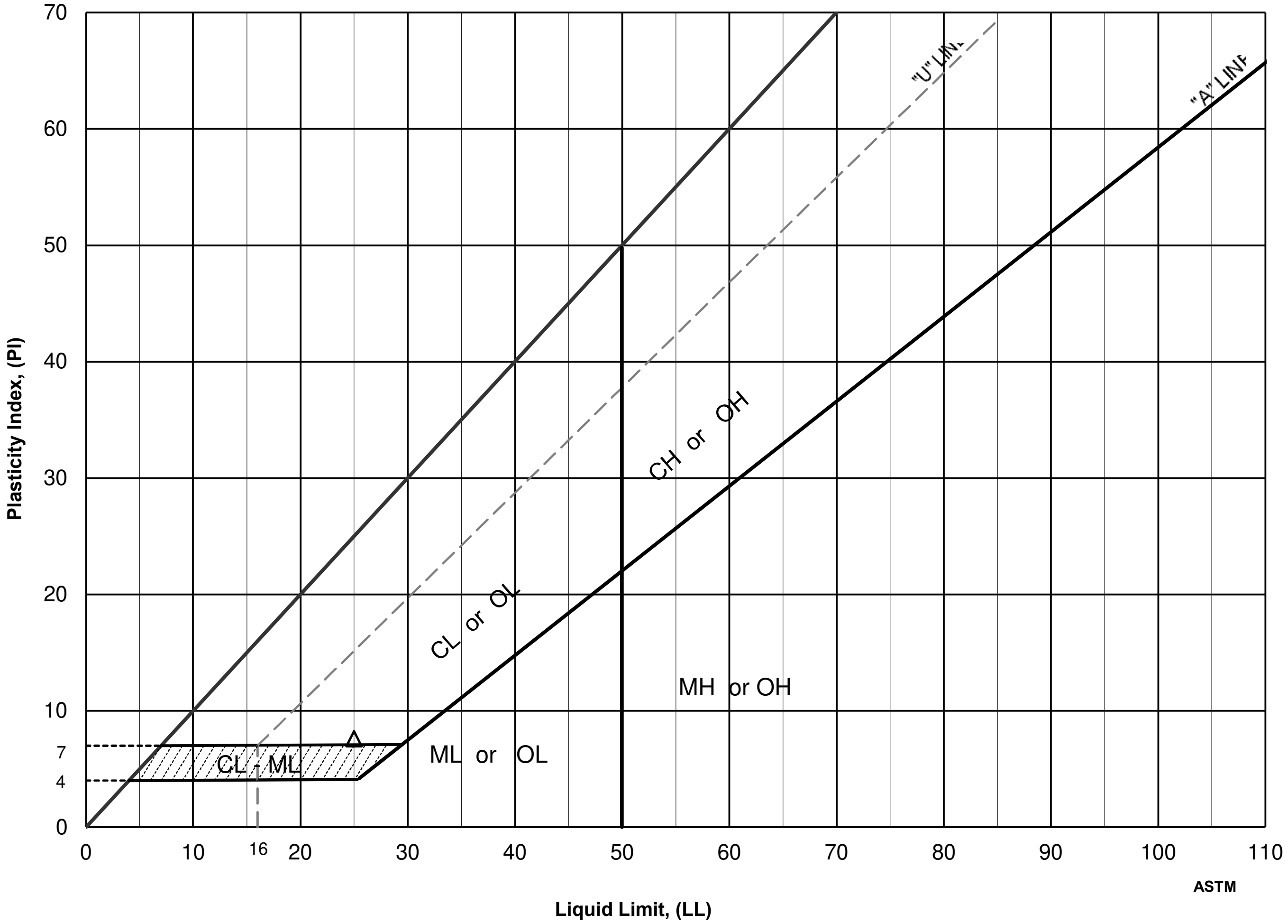
LRL Associates Ltd.

PLASTICITY INDEX

ASTM D 4318 / LS-703/704

Client: Halo Car Wash Inc.
File No.: 210691
Project: Geotechnical Investigation
Report No.: 1
Location: 3555 Borriskane Road, Nepean, ON.
Date: February 9, 2022

Plasticity Chart



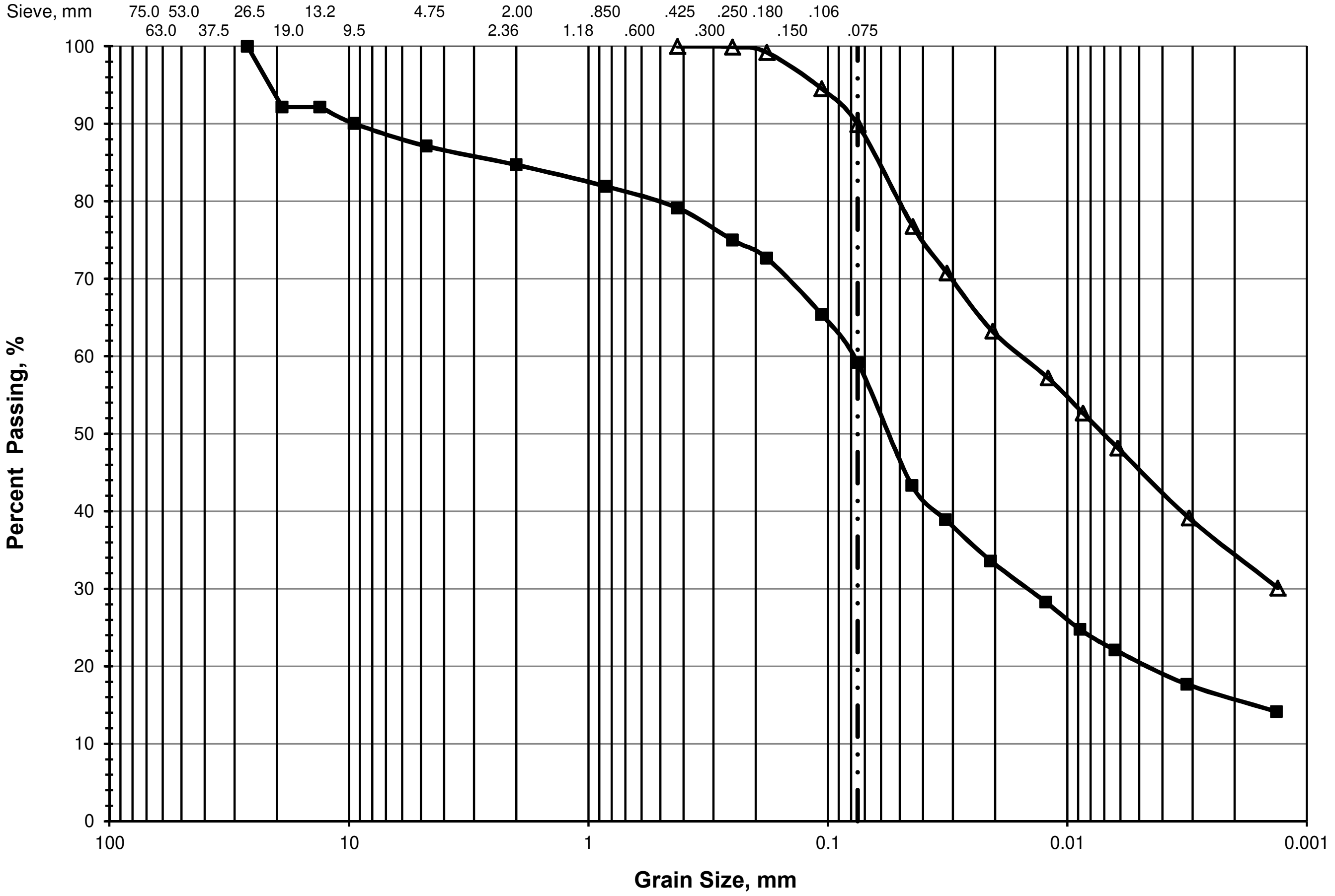
	Location	Sample	Depth, m	Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Activity Number	USCS
△	BH 1	SS-4	2.29 - 2.90	32	25	17	8	1.89	n/d	CL

LRL Associates Ltd.

PARTICLE SIZE ANALYSIS

ASTM D 422 / LS-702

Client: Halo Car Wash Inc.
Project: Geotechnical Investigation
Location: 3555 Borrisokane Road, Nepean, ON.
File No.: 210691
Report No.: 2
Date: February 9, 2022



Unified Soil Classification System

	> 75 mm	% GRAVEL		% SAND			% FINES	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
△	0.0	0.0	0.0	0.0	0.0	10.1	56.3	33.6
■	0.0	6.8	6.1	2.4	5.5	20.0	43.8	15.4

	Location	Sample	Depth, m	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
△	BH 3	SS-7	6.10 - 6.71	0.0160	0.0072	0.0013				
■	BH 4	SS-3	1.52 - 2.13	0.0792	0.0574	0.0151	0.0018			

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Certificate of Analysis

LRL Associates Ltd.

5430 Canotek Road
Ottawa, ON K1J 9G2
Attn: Brad Johnson

Client PO:
Project: 210691
Custody: 64617

Report Date: 17-Feb-2022
Order Date: 14-Feb-2022

Order #: 2208056

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
2208056-01	BH4 5-7'

Approved By: Mark Foto

Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: LRL Associates Ltd.
Client PO:

Report Date: 17-Feb-2022
Order Date: 14-Feb-2022
Project Description: 210691

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	16-Feb-22	16-Feb-22
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	14-Feb-22	15-Feb-22
Resistivity	EPA 120.1 - probe, water extraction	16-Feb-22	17-Feb-22
Solids, %	Gravimetric, calculation	15-Feb-22	15-Feb-22

Certificate of Analysis

Client: **LRL Associates Ltd.**

Client PO:

Report Date: 17-Feb-2022

Order Date: 14-Feb-2022

Project Description: 210691

	Client ID:	BH4 5-7'	-	-	-
	Sample Date:	09-Feb-22 09:00	-	-	-
	Sample ID:	2208056-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	82.7	-	-	-
General Inorganics					
pH	0.05 pH Units	7.59	-	-	-
Resistivity	0.10 Ohm.m	42.4	-	-	-
Anions					
Chloride	5 ug/g dry	30	-	-	-
Sulphate	5 ug/g dry	84	-	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	8.5	5	ug/g dry	8.5			0.5	20	
Sulphate	53.4	5	ug/g dry	54.2			1.4	20	
General Inorganics									
pH	7.38	0.05	pH Units	7.40			0.3	2.3	
Resistivity	159	0.10	Ohm.m	151			4.7	20	
Physical Characteristics									
% Solids	82.1	0.1	% by Wt.	86.4			5.1	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	102	5	ug/g	8.5	93.5	82-118			
Sulphate	153	5	ug/g	54.2	98.4	80-120			

Qualifier Notes:

None

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.