FINAL

Geotechnical Investigation – Proposed Building Addition

1726 Baseline Road West, Bowmanville, Ontario

Prepared for:

Cestoil Chemical Inc.
175 West Beaver Creek Rd, Unit 9
Richmond Hill, Ontario, L4N 3M1

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1.0 INTRODUCTION AND SCOPE

Pinchin Ltd. (Pinchin) was retained by Cestoil Chemical Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed building addition to be located at 1726 Baseline Road West, Bowmanville, Ontario (Site). The Site location is shown on Figure 1.

The Site is currently developed with a one storey warehouse building (Site Building). Based on information provided by the Client, which included the document entitled, *"Proposed Industrial Addition, 1726 Baseline Road, Clarington (Courtice), Ontario"*, prepared by Vincent J. Santamaura Architect Inc., dated April 30, 2025, it is Pinchin's understanding that the proposed development is to consist of a single-storey, slab-on-grade (i.e. no basement level) addition on the south side of the existing building. The addition will have an area of about 845 square meters and a finished floor elevation of 98.58 meters above mean sea level, which matches the finished floor level of the existing building.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of three (3) sampled boreholes (Boreholes BH-01 to BH-03), at the Site.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A detailed description of the soil and groundwater conditions;
- Site preparation recommendations;
- Open cut excavations;
- Anticipated groundwater management;
- Foundation design recommendations including soil bearing resistances at Ultimate Limit States (ULS) and Serviceability Limit States (SLS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Concrete floor slab-on-grade support recommendations; and,
- Potential construction concerns.

Abbreviations terminology and principle symbols commonly used throughout the report, borehole logs and appendices are enclosed in Appendix I.

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is located on the north side of Baseline Road, approximately 200 m west of Courtice Road , in Bowmanville, Ontario. The Site is currently developed with a single-storey warehouse building located in the north portion of the Site. The lands adjacent to the Site are developed with commercial buildings.

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on a fine textured glaciomarine deposit consisting of massive to well laminated silt and clay with minor sand and gravel deposits. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey 2011. 1:250 000 scale bedrock geology of Ontario; Ontario Geological Survey, Miscellaneous Release---Data 126-Revision 1).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed field investigations at the Site on September 24, 2025, by advancing a total of three (3) sampled boreholes (BH-01 to BH-03) throughout the Site. The boreholes were advanced to depths of approximately 6.7 metres below existing ground surface (mbgs). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 3230 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.75 to 1.5 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil. Approximate shear strengths of the cohesive deposits were measured using a handheld pocket penetrometer and the results are presented on the appended borehole logs.

The borehole locations and ground surface elevations were surveyed by Pinchin using a Trimble Catalyst Global Navigation Satellite System (GNSS) RTK rover. The ground surface elevations are geodetic, based on GNSS and local base station telemetry with a precision static of less than 20 mm. The survey was completed to Projection UTM-17, Datum: NAD 83, Geoid: HT2_0.

It is recommended that a licensed land surveyor confirm the elevations of the borehole locations, if the elevations are to be used for any calculation purposes.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to Pinchin’s accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples collected from the boreholes were submitted to Pinchin's material testing laboratory to determine the moisture content, grain size distribution and Atterberg Limits of the soil. Copies of the laboratory analytical reports are included in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy

In general, the soil stratigraphy at the Site comprises fill and possible fill overlying native deposits of silty clay / clayey silt till which in turn overlied sandy silty clay till to the maximum borehole termination depths of approximately 6.7 mbgs. The appended borehole logs provide detailed soil descriptions and stratigraphies, results of SPT and pocket penetrometer testing, and moisture content profiles.

A layer of fille fill was encountered in Boreholes BH-02 and BH-03, with thickness of approximately 0.8 m. Possible fill was encountered at ground surface at BH-01, extending to about 0.8 mbgs; and, below the fill at BH-02, extending to about 1.5 mbgs. The fill comprised sand and gravel or coarse sand at surface, underlain by grey clayey silt with trace sand and nil to trace gravel SPT 'N' values in the fill and possible fill ranged from 6 to 22 blows per 300 mm, indicating the non-cohesive portions are compact, and the cohesive portions are firm.

The moisture content of the samples of fil land possible fill tested ranged from 3.7% to 22.9% indicating that the non-cohesive fill was damp to moist, and the cohesive fill and possible fill was Wetter Than Plastic Limit (WTPL).

Glacial till was encountered underlying the fill within all boreholes and extended to the borehole termination depths of about 6.7 mbgs (90.5 to 90.8 masl). The glacial till generally comprised clayey silt / silty clay overlying sandy silty clay.

The upper glacial till consisted of brown mottled to grey silty clay / clayey silt till and generally contained trace sand and gravel. Traces of organics and oxidation patches was noted in the upper portions of this deposit. The glacial till had a stiff to very stiff consistency based on SPT ‘N’ values of 14 to 22 blows per 300 mm penetration of a split spoon sampler. Shear strengths measured with a handheld pocket penetrometer of 125 to 200 kPa. Moisture contents of the samples ranged between 19 to 22%. The upper glacial till was described as being WTPL at time of sampling.

The lower portion of the glacial till comprised sandy silty clay till. The material was brown to grey in colour, with trace to some gravel, trace wet seams. The cohesive glacial till had a firm to very stiff consistency based on SPT ‘N’ values of 6 to 26 blows per 300 mm penetration of a split spoon sampler. The relative density of the soil was generally observed to decrease with depth, with the deposits being consistently stiff to firm more below elevation 94 masl.

Shear strengths measured with a handheld pocket penetrometer of 100 to greater than 200 kPa. Sample SS4, collected from a depth of 2.3 to 2.9 mbgs in borehole BH-01, identified 6% gravel, 30% sand, 46% silt, and 18% clay based on the results of particle size distribution analysis, provided in Appendix III. Atterberg limit testing on this sample indicated a liquid limit of 17%, a plastic limit of 10%, and a plasticity index of 7%. Moisture contents of the glacial till samples ranged between 9 to 12% indicating that the material was About the Plastic Limit (APL) at time of sampling.

4.2 Groundwater Conditions

No groundwater levels were observed in the boreholes at the time of drilling. Typically, the grey colour of the soils noted in the boreholes between Elevation 94.5 and 96.4 masl is indicative of permanent saturated conditions, and therefore, the fluctuations of the long-term groundwater should not be expected to drop below this depth. It is noted that groundwater could be locally perched in sandy fill deposits overlying less permeable fill or native soils. Perched groundwater may occur above these depths particularly following heavy rainfall or snowmelt.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the geotechnical investigation, and Pinchin's experience with similar projects. Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

It is Pinchin's understanding that the development will consist of a single-storey slab-on-grade (i.e. no basement level) warehouse addition to the south of the existing building located at the Site. It is understood that the finished floor level of the addition will match the finished floor level of the existing building.

5.2 Site Preparation

The existing fill is not considered suitable to remain below the proposed building foundations and floor slabs, driveways and parking areas and will need to be removed. The deposits noted as possible fill should be investigated further during construction. If they are found to be fill, they will also need to be removed from below the proposed building.

Prior to placing any soils at the Site, the subgrade should be inspected by a qualified geotechnical engineer, and any loosened/soft pockets should be sub-excavated. Imported inorganic soil can be used as engineered fill provided that it can be placed and compacted as per the criteria stated in the following table. The imported soil should free of debris and deleterious material.

Pinchin recommends that any engineered fill required at the Site be compacted in accordance with the criteria stated in the following table:

Type of Engineered Fill	Maximum Loose Lift Thickness (mm)	Compaction Requirements	Moisture Content (Percent of Optimum)
Structural fill to support foundations and floor slabs	200	100% SPMDD	Plus 2 to minus 4
Subgrade fill beneath parking lots and access roadways	300	98% SPMDD	Plus 2 to minus 4

Any additional fill needed could comprise imported Ontario Provincial Standard Specification (OPSS) 1010 Granular 'B' or Select Subgrade Material (SSM). If the work is carried out during very dry weather, water may have to be added to the material to improve compaction.

The structural fill pad should extend at least 1.0 metre beyond the footing edge of any building and outwards and downwards to the subgrade level at a slope of 1.0 horizontal to 1.0 vertical.

A sheepsfoot compactor should be used at sites on cohesive material. A smooth drum roller should be utilized where imported sandy soils are used as fill. The number of passes required will vary depending on the equipment used, fill material type, and moisture condition of the fill. The compaction should be verified by in-situ density testing.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

The above noted recommendations are from a geotechnical perspective and additionally analytical requirements may need to be reviewed in order to ensure compliance with Ontario Regulation 406/19, *On-Site and Excess Soil Management,* depending on when the material is received at the Site.

5.3 Open Cut Excavations

It is anticipated that the foundations will be constructed at conventional frost depths, approximately 1.5 metres below finished floor elevation. Based on the subsurface information obtained from within the boreholes, it is anticipated that the excavated material will predominately consist of fill material and native glacial till. At the completion of drilling, groundwater levels were not observed in the open boreholes at the time of drilling.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). The use of trench boxes can most likely be used for temporary support of vertical side walls. The appropriate trench should be designed/confirmed for use in this soil deposit.

Based on the OHSA, the fill material would be classified as Type 3 soil and temporary excavations in these soils must be sloped at an inclination of 1 horizontal to 1 vertical (H to V) from the base of the excavation. Excavations extending below the groundwater table, if encountered, would be classified as

Type 4 soil. Temporary excavations in such soils must be sloped back at a gradient of 3H:1V from the base of the excavation. Excavations through more than one soil type must be made in conformance with the requirements for the soil type with the highest number.

In addition to compliance with the OHSA, the excavation procedures must also be in compliance to any potential other regulatory authorities, such as federal and municipal safety standards.

Alternatively, the excavation walls may be supported by either closed shoring, or bracing, complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). Pinchin would be pleased to provide further recommendations on shoring design once the building plans have been completed.

5.4 Anticipated Groundwater Management

Groundwater was not observed in the boreholes during drilling and is not anticipated to be encountered during excavations for the building foundations; however, it is noted that groundwater could be locally perched at shallower depths in sandy fill deposits overlying less permeable fill or native soils.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential precipitation or perched groundwater should be able to be controlled from pumping from filtered sumps.

Prior to commencing excavations, it is critical that all existing surface water and potential surface water is controlled and diverted away from the Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to precipitation and cause subgrade softening.

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential impacts on the environment.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. Excavations to conventional design depths for the building foundations are not expected to require a Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR). It is the responsibility of the contractor to make this application if required.

As previously mentioned, above average seasonal variations in the groundwater table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. As such, depending on the groundwater at the time of the excavation works, a more involved dewatering system may be required.

5.5 Foundation Design

5.5.1 Shallow Foundations Bearing on Glacial Till or Engineered Structural Filll

The existing native glacial till is considered suitable to support the proposed building, provided all of the fill is removed, and the subgrade prepared as above. Where, following removal of unsuitable soils, the level of the suitable native soils are below the design underside of footing level, grades can be raised with engineered structural fill as noted in Section 5.2 of this report.

Conventional shallow strip footings established on the undisturbed glacial till or approved engineered fill, placed and compacted as described in Section 5.2, may be designed using a bearing resistance for 25 mm of settlement at Serviceability Limit States of 150 kPa, and a factored geotechnical bearing resistance of 225 kPa at Ultimate Limit States (ULS). These values are limited to footing sizes of up to 2.0 m in width for strip footings and 2.5 m in width for pad footings.

It is noted that higher bearing pressures may be available for footings established on the very stiff glacial till at about 2.3 m below existing grades. Pinchin can provide more info on the potential for higher bearing pressures at these levels, if desired.

As the actual service loads were not known at the time of this report, these should be reviewed by the project structural engineer to determine if SLS or ULS governs the footing design.

Where the footings of the proposed addition abut the footings of the existing building, the footing levels should match in order to avoid uneven stress distributions and/or undermining.

It is noted that there is a potential for weaker subgrade soil to be encountered between the investigation locations. Pinchin presumes that any areas of weaker subgrade soil will consist of small pockets of soft/loose natural soil which can be compacted to match the density of the remainder of the Site. As such, the material must be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD) prior to installing the concrete formwork. Any soft/loose areas which are not able to achieve the recommended 100% SPMDD are to be removed and replaced with a low strength concrete.

Pinchin notes that a qualified geotechnical engineering consultant should be on-Site during the proof roll and foundation preparation activities to verify the recommended level of compaction is achieved and to verify the design assumptions and recommendations. This is especially critical with respect to the recommended soil bearing pressures. If variations occur in the soil conditions between the borehole locations, site verification and site review by Pinchin is recommended to provide appropriate recommendations at that time.

The natural subgrade soil is sensitive to change in moisture content and can become loose/soft if subjected to additional water or precipitation. As well, it could be easily disturbed if travelled on during construction. Once it becomes disturbed it is no longer considered adequate to support the recommended design bearing pressures. It is recommended that a working slab of lean concrete (mud slab) be placed in the footing areas immediately after excavation and inspection to protect the founding soils during placement of formwork and reinforcing steel.

In addition, to ensure and protect the integrity of the subgrade soil during construction operations, the following is recommended:

- Prior to commencing excavations, it is critical that all existing surface water, potential surface water and perched groundwater are controlled and diverted away from the work Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to inclement weather conditions and cause subgrade softening;
- The subgrade should be sloped to a sump outside the excavation to promote surface drainage and the collected water pumped out of the excavation. Any potential precipitation or seepage entering the excavations should be pumped away immediately (not allowed to pond);
- The footing areas should be cleaned of all deleterious materials such as organics, fill, disturbed, or caved materials;
- Any potential large cobbles or boulders (i.e. greater than 200 mm in diameter) within the subgrade material are to be removed and replaced with a similar soil type not containing particles greater than 200 mm in diameter. It is critical that particles greater than 200 mm in diameter are not in contact with the foundation to prevent point loading and overstressing; and
- If the excavated subgrade soil remains open to weather conditions and groundwater seepage, sidewall stability and suitability of the subgrade soil will need to be verified prior to construction.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing bases and concrete must be provided and maintained above freezing at all times.

5.5.2 Estimated Settlement

All individual spread footings should be founded on uniform subgrade soils, reviewed and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the 2024 OBC.

5.5.3 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

Exterior perimeter foundations drains are not required, where the finished floor elevation is established a minimum of 150 mm above the exterior final grades or that the exterior gradient is properly sloped to divert surface water away from the building.

5.5.4 Shallow Foundations Frost Protection & Foundation Backfill

In the Bowmanville, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.2 m of soil cover above the underside of the footing to provide soil cover for frost protection.

Where the foundations for heated buildings do not have the minimum 1.2 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular ‘B’ Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The existing silt material is not considered suitable for reuse as foundation wall backfill. Backfill must be brought up evenly on both sides of foundation walls not designed to resist lateral earth pressure. All backfill material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD below the interior of building and exterior hard landscaping areas; and 95% SPMDD below exterior soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6 Floor Slabs

Prior to the installation of the engineered fill material, all organics and deleterious materials should be removed to the underlying organic free in-situ soil as described in Section 5.2. The natural subgrade soil is to be proof roll compacted with a minimum 10 tonne non-vibratory steel drum roller to observe for weak/soft spots. It is noted that some locations will not be accessible by the steel drum roller; as such, these locations can be proof roll compacted with a minimum 450 kg vibratory plate compactor.

The in-situ inorganic till encountered within the boreholes is considered adequate for the support of the concrete floor slabs provided it is proof roll compacted as outlined above. Any soft area(s) encountered during proof rolling should be excavated and replaced with a similar soil type.

Once the subgrade soil is exposed it is to be inspected and approved by a qualified geotechnical engineering consultant to ensure that the material conforms to the soil type and consistency observed during the subsurface investigation work.

Based on the in-situ soil conditions, it is recommended to establish the concrete floor slab on a minimum 300 mm thick layer of Granular “A” (OPSS 1010) compacted to 100% SPMDD. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone placed over the approved subgrade. Any required-up fill should consist of a Granular “B” Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturers and designer’s requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.

Subgrade support for concrete floor on ground is measured by Westergaard’s modulus of subgrade reaction, k. Provided that the floor slab is constructed directly on 200 mm of crushed gravel base an approximate value of k for the soil at the Site is 25,000 kN/m^3. The modulus of subgrade reaction would be lower for the Site if the floor slab is not constructed as noted above. The value is for loaded areas of 0.3 m by 0.3 m.

5.7 Site Classification for Seismic Site Response & Soil Behaviour

The following information has been provided to assist the building designer from a geotechnical perspective only. These geotechnical seismic design parameters should be reviewed in detail by the structural engineer and be incorporated into the design as required.

The seismic site classification has been based on the 2024 OBC or 2020 NBCC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.B of the OBC/NBCC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy. If the average shear wave velocity is not known, the site class can be estimated from energy corrected Standard Penetration Resistance (N60) and/or the average undrained shear strength of the soil in the top 30 m.

The boreholes advanced at this Site extended to approximately 7.6 mbgs and were terminated in the soil deposit. SPT “N” values within the soil deposit ranged between 6 and 26 blows per 300 mm. As such, based on Table 4.1.8.4.B of the OBC/NBCC, this Site has been classified as Class D. A Site Class D has an average shear wave velocity (Vs) of between 180 and 360 m/s.

5.8 Asphaltic Concrete Pavement Structure Design for Parking Lot and Driveways

5.8.1 Discussion

Parking areas and driveway access will be constructed around the proposed buildings. The in-situ silty clayey till is considered a sufficient bearing material for an asphaltic concrete pavement structure provided all organics and deleterious materials are removed prior to installing the engineered fill material.

For the purpose of this report, it has been assumed that the subgrade will be prepared according to Section 5.2 and will comprise approved onsite soils or OPSS 1010 Granular B or SSM engineered fill.

5.8.2 Pavement Structure

No traffic design details were available at the time of this report. It is recommended that Pinchin review the provided pavement design recommendations once additional traffic design detail becomes available.

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Parking Areas	Driveways
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	92% MRD as per OPSS 310	40 mm	40 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92% MRD as per OPSS 310	50 mm	80 mm
Base Course: Granular “A” (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular “B” Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	450 mm

Notes:
Prior to placing the pavement structure, the subgrade soil is to be proof rolled with a smooth drum roller without vibration to observe weak spots and the deflection of the soil; and

The recommended pavement structure may have to be adjusted according to the City of Bowmanville standards. Also, if construction takes place during times of substantial precipitation and the subgrade soil becomes wet and disturbed, the granular thickness may have to be increased to compensate for the weaker subgrade soil. In addition, the granular fill material thickness may have to be temporarily increased to allow heavy construction equipment access the Site, in order to avoid the subgrade from "pumping" up into the granular material.

Performance grade PG 58-28 asphaltic concrete should be specified for Marshall mixes. Consideration should be given to increasing the PGAC grade to 64-28 for any areas of high truck traffic.

5.8.3 Pavement Structure Subgrade Preparation and Granular up Fill

The proper placement of base and subbase fill materials becomes very important in addressing the proper load distribution to provide a durable pavement structure.

The pavement subgrade materials should be thoroughly proof rolled prior to placement of the Granular 'B' subbase course. If any unstable areas are noted, then the Granular 'B' thickness may need to be increased to support pavement construction traffic. This should be left as a field decision by a qualified geotechnical engineer at the time of construction, but it is recommended that additional Granular 'B' be carried as a provisional item under the construction contract.

Where fill material is required to increase the grade to the underside of the pavement structure it should consist of Granular 'B' Type I (OPSS 1010). The up-fill material is to be placed in maximum 300 mm thick lifts compacted to 98% SPMDD within 4% of the optimum moisture content.

Samples of both the Granular 'A' and Granular 'B' Type I aggregates should be tested for conformance to OPSS 1010 prior to utilization on Site and during construction. All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Post compaction settlement of fine-grained soil can be expected, even when placed to compaction specifications. As such, fill material should be installed as far in advance as possible before finishing the parking lot and access roadways for best grade integrity.

Where the subgrade material types differ below the underside of the pavement structure, the transition between the materials should be sloped as per frost heave taper OPSD 205.60.

5.8.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches.

The silty /clayey soils have poor natural drainage and therefore it is recommended that pavement subdrains be installed in the lower areas and be connected to the catch basins. Pavement subdrains should comprise 150 mm diameter perforated pipe in filter sock, bedded in concrete sand. The upper limit of the subdrain bedding should be at the lower limit of the pavement subbase, with the subgrade below the subbase sloped towards the subdrain.

The surface of the roadways should be free of depressions and be sloped at a minimum grade of 1% in order to drain to appropriate drainage areas. Subgrade soil should slope a minimum of 3% toward stormwater collection points. Positive slopes are very important for the proper performance of the drainage system. The granular base and subbase materials should extend horizontally to any potential ditches or swales.

In addition, routine maintenance of the drainage systems will assist with the longevity of the pavement structure. Ditches, culverts, sewers and catch basins should be regularly cleared of debris and vegetation.

6.0 SITE SUPERVISION & QUALITY CONTROL

It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the undisturbed natural subgrade material prior to subgrade preparation, pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin's recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

7.0 TERMS AND LIMITATIONS

This Geotechnical Investigation was performed for the exclusive use of Cestoil Chemical Inc. (Client) in order to evaluate the subsurface conditions at 1726 Baseline Road West, Bowmanville, Ontario. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practises in the field of geotechnical engineering for the Site. Classification and identification of soil, and geologic units have been based upon commonly accepted methods employed in professional geotechnical practice. No warranty or other conditions, expressed or implied, should be understood. Conclusions derived are specific to the immediate area of study and cannot be extrapolated extensively away from sample locations.

Performance of this Geotechnical Investigation to the standards established by Pinchin is intended to reduce, but not eliminate, uncertainty regarding the subgrade soil at the Site, and recognizes reasonable limits on time and cost.

Regardless how exhaustive a Geotechnical Investigation is performed; the investigation cannot identify all the subsurface conditions. Therefore, no warranty is expressed or implied that the entire Site is representative of the subsurface information obtained at the specific locations of our investigation. If during construction, subsurface conditions differ from then what was encountered within our test location and the additional subsurface information provided to us, Pinchin should be contacted to review our recommendations. This report does not alleviate the contractor, owner, or any other parties of their respective responsibilities.

This report has been prepared for the exclusive use of the Client and their authorized agents. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

The liability of Pinchin or our officers, directors, shareholders or staff will be limited to the lesser of the fees paid or actual damages incurred by the Client. Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered (Claim Period), to commence legal proceedings against Pinchin to recover such losses or damage unless the laws of the jurisdiction which governs the Claim Period which is applicable to such claim provides that the applicable Claim Period is greater than two years and cannot be abridged by the contract between the Client and Pinchin, in which case the Claim Period shall be deemed to be extended by the shortest additional period which results in this provision being legally enforceable.

Pinchin makes no other representations whatsoever, including those concerning the legal significance of its findings, or as to other legal matters touched on in this report, including, but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and these interpretations may change over time. Please refer to Appendix IV, Report Limitations and Guidelines for Use, which pertains to this report.

Specific limitations related to the legal and financial and limitations to the scope of the current work are outlined in our proposal, the attached Methodology and the Authorization to Proceed, Limitation of Liability and Terms of Engagement which accompanied the proposal.

Information provided by Pinchin is intended for Client use only. Pinchin will not provide results or information to any party unless disclosure by Pinchin is required by law. Any use by a third party of reports or documents authored by Pinchin or any reliance by a third party on or decisions made by a third party based on the findings described in said documents, is the sole responsibility of such third parties. Pinchin accepts no responsibility for damages suffered by any third party as a result of decisions made or actions conducted. No other warranties are implied or expressed.

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Template: Master Geotechnical Investigation Report – Ontario, GEO, January 23, 2025

FIGURES

PINCHIN	PROJECT NAME: GEOTECHNICAL INVESTIGATION				
	CLIENT NAME: CESTOIL CHEMICAL INC.				
	PROJECT LOCATION: 1726 BASELINE ROAD WEST, BOWMANVILLE, ONTARIO				
	FIGURE NAME: KEY MAP				FIGURE NUMBER: 1
	PROJECT NUMBER: 361920	SCALE: 1:5,000	DRAWN BY: KM	REVIEWED BY: NN	DATE: OCTOBER 2025

N

LEGEND

SITE BOUNDARY

SITE BUILDING

BOREHOLE

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PINCHIN

PROJECT NAME: GEOTECHNICAL INVESTIGATION	
CLIENT NAME: CESTOIL CHEMICAL INC.	
PROJECT LOCATION: 1726 BASELINE ROAD WEST, BOWMANVILLE, ONTARIO	
FIGURE NAME: BOREHOLE LOCATION PLAN	
PROJECT NUMBER: 361920	SCALE: AS SHOWN
DRAWN BY: KM	REVIEWED BY: NN
DATE: OCTOBER 2025	FIGURE NUMBER: 2

APPENDIX I

Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

AS	Auger Sample	**w**	Washed Sample
SS	Split Spoon Sample	**HQ**	Rock Core (63.5 mm diam.)
ST	Thin Walled Shelby Tube	**NQ**	Rock Core (47.5 mm diam.)
BS	Block Sample	**BQ**	Rock Core (36.5 mm diam.)

In-Situ Soil Testing

Standard Penetration Test (SPT), "N" value is the number of blows required to drive a 51 mm outside diameter split spoon barrel sampler into the soil a distance of 300 mm with a 63.5 kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved. The SPT, "N" value is a qualitative term used to interpret the compactness condition of cohesionless soils and is used only as a very approximation to estimate the consistency and undrained shear strength of cohesive soils.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60-degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm2 base area with a 60 degree apex pushed through the soil at a penetration rate of 2 cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

Soil Descriptions

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm		
Silt	0.002 to 0.06 mm	"trace", trace sand, etc.	1 to 10%
Sand	0.075 to 4.75 mm	"some", some sand, etc.	10 to 20%

Soil Classification		Terminology	Proportion
Gravel	4.75 to 75 mm	Adjective, sandy, gravelly, etc.	20 to 35%
Cobbles	75 to 200 mm	And, and gravel, and silt, etc.	>35%
Boulders	>200 mm	Noun, Sand, Gravel, Silt, etc.	>35% and main fraction

Notes:

- Soil properties, such as strength, gradation, plasticity, structure, dictate the soils engineering behaviour over grain size fractions; and
- With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations. The accuracy of visual and tactile observation is not sufficient to differentiate between changes in soil classification or precise grain size and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the compactness condition of cohesionless soil:

Cohesionless Soil	
Compactness Condition	**SPT N-Index (blows per 300 mm)**
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soil		
Consistency	**Undrained Shear Strength (kPa)**	**SPT N-Index (blows per 300 mm)**
Very Soft	<12	<2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30

Cohesive Soil		
Consistency	**Undrained Shear Strength (kPa)**	**SPT N-Index (blows per 300 mm)**
Hard	>200	>30

Note: Utilizing the SPT, N-Index value to correlate the consistency and undrained shear strength of cohesive soils is only very approximate and needs to be used with caution.

Soil & Rock Physical Properties

General

W Natural water content or moisture content within soil sample

γ Unit weight

γ' Effective unit weight

γ_d Dry unit weight

γ_{sat} Saturated unit weight

ρ Density

ρ_s Density of solid particles

ρ_w Density of Water

ρ_d Dry density

ρ_{sat} Saturated density

e Void ratio

n Porosity

S_r Degree of saturation

E_{50} Strain at 50% maximum stress (cohesive soil)

Consistency

W_L Liquid limit

W_P Plastic Limit

I_P Plasticity Index

W_S Shrinkage Limit

I_L Liquidity Index

I_C	Consistency Index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density Index (formerly relative density)

Shear Strength

C_u, S_u	Undrained shear strength parameter (total stress)
C'_d	Drained shear strength parameter (effective stress)
r	Remolded shear strength
τ_p	Peak residual shear strength
τ_r	Residual shear strength
ø'	Angle of interface friction, coefficient of friction = tan ø'

Consolidation (One Dimensional)

C_C	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
C_S	Swelling index
m_V	Coefficient of volume change
c_V	Coefficient of consolidation
T_V	Time factor (vertical direction)
U	Degree of consolidation
σ'_o	Overburden pressure
σ'_p	Preconsolidation pressure (most probable)
OCR	Overconsolidation ratio

Premeability

The following table outlines the terms used to describe the degree of permeability of soil and common soil types associated with the permeability rates:

Permeability (k cm/s)	Degree of Permeability	Common Associated Soil Type
> 10^{-1}	Very High	Clean gravel
10^{-1} to 10^{-3}	High	Clean sand, Clean sand and gravel
10^{-3} to 10^{-5}	Medium	Fine sand to silty sand
10^{-5} to 10^{-7}	Low	Silt and clayey silt (low plasticity)
>10^{-7}	Practically Impermeable	Silty clay (medium to high plasticity)

Rock Coring

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum.

RQD is calculated as follows:

$$\text{RQD (\%)} = \frac{\Sigma \text{ Length of core pieces} > 100 \text{ mm x } 100}{\text{Total length of core run}}$$

The following is the Classification of Rock with Respect to RQD Value:

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

Document1

Template: Appendix – Abbreviations Terminology & Principal Symbols, GEO, January 21, 2025

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: BH-01

Project #: 361920 **Logged By:** CG

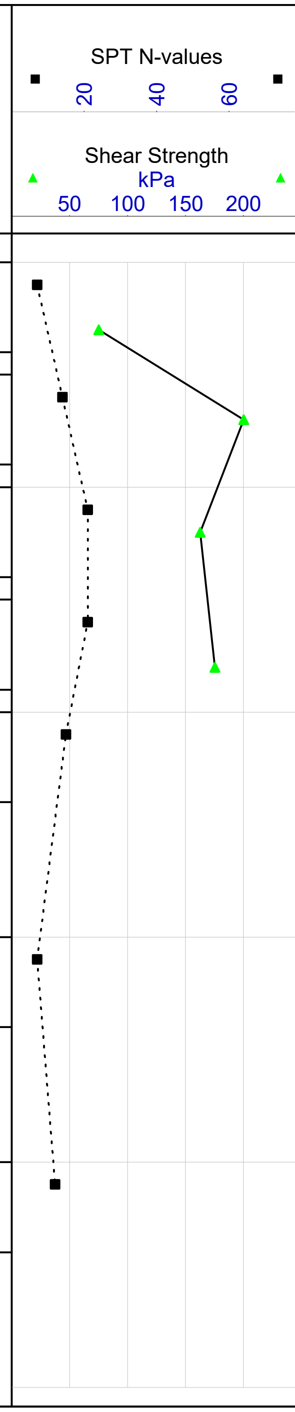
Project: Geotechnical Investigation

Client: Cestoil Chemical Inc.

Location: 1726 Baseline Road West, Bowmanville, Ontario

Drill Date: September 24, 2025 **Project Manager:** NN

SUBSURFACE PROFILE				SAMPLE						
Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Soil Vapour Concentration
Ground Surface	97.22	No Monitoring Well Installed							
Possible Fill Brown gravel and sand, 100 mm, grey clayey silt, trace sand, trace organics, trace rootlets, firm, WTPL.	96.46		SS	1	100	7		22.9	
Clayey Silt Till Grey brown, trace sand and gravel, trace oxidation patches, stiff, WTPL	95.70		SS	2	90	14		22.7	
Mottled, very stiff.	94.94		SS	3	90	21		18.8	
Sandy Silty Clay Till Light brown, trace gravel, very stiff, APL.	94.17		SS	4	100	21		9.9	
Some gravel, wet seams.	92.65		SS	5	100	15		10.0	
Firm, APL.	91.13		SS	6	50	7		11.6	
Stiff.	90.52		SS	7	100	12		11.9	
End of Borehole									



Contractor: Strata Drilling Group **Grade Elevation:** 97.22 masl

Drilling Method: Direct Push **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH-02

Project #: 361920 **Logged By:** CG

Project: Geotechnical Investigation

Client: Cestoil Chemical Inc.

Location: 1726 Baseline Road West, Bowmanville, Ontario

Drill Date: September 24, 2025 **Project Manager:** NN

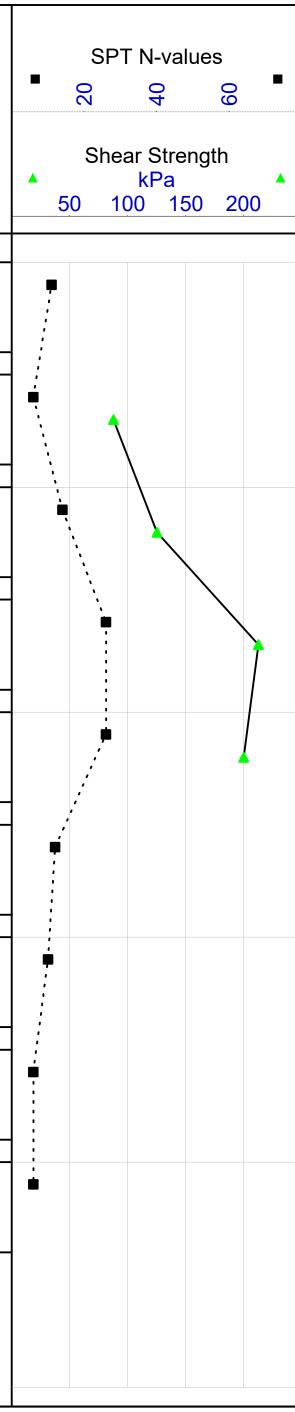
SUBSURFACE PROFILE

Depth (m)	Description	Elevation (m)
0	Ground Surface	97.38
0 – 0.76	**Fill** Brown sand and gravel to 100 mm, Brown coarse sand, compact, moist.	96.62
0.76 – 1.52	**Possible fill** Brown/ grey Mottled silty clay, trace organic seams, trace sand and gravel, firm, WTPL	95.86
1.52 – 2.29	**Silty Clay Till** Brown/ grey, trace sand and gravel, with oxidation patched, stiff, APL.	95.09
2.29 – 3.81	**Sandy Silty Clay Till** Light grey, trace gravel, very stiff, APL.	93.57
3.81 – 6.71	trace to some gravel, wet seams, stiff.	90.67
6.71	End of Borehole	

Monitoring Well Details: No Monitoring Well Installed

SAMPLE

Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Soil Vapour Concentration
SS	1	50	11		7.2	
SS	2	80	6		21.9	
SS	3	100	14			
SS	4	100	26		9.4	
SS	5	100	26		9	
SS	6	90	12		9.6	
SS	7	70	10		10.1	
SS	8	50	6		10.2	
SS	9	100	6		10.4	



Contractor: Strata Drilling Group **Grade Elevation:** 97.38 masl

Drilling Method: Direct Push **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH-03

Project #: 361920 **Logged By:** CG

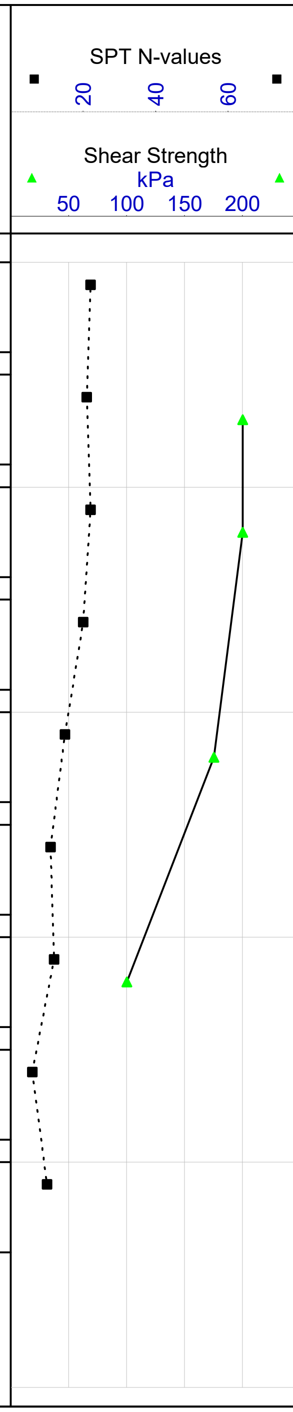
Project: Geotechnical Investigation

Client: Cestoil Chemical Inc.

Location: 1726 Baseline Road West, Bowmanville, Ontario

Drill Date: September 24, 2025 **Project Manager:** NN

Depth (m)	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-values	Lab Analysis	Moisture (%)	Soil Vapour Concentration
0	Ground Surface	97.49	No Monitoring Well Installed							
0–0.76	**Fill** Brown sand and gravel, compact, moist.	96.73		SS	1	100	22		3.7	
0.76–2.28	**Silty Clay Till** Brown Mottled, trace organics, trace sand and gravel, very stiff, WTPL	95.21		SS	2	90	21		18.7	
				SS	3	90	22		22	
2.28–3.04	**Sandy Silty Clay Till** Brown, trace to some gravel, very stiff, APL.	94.45		SS	4	100	20		10.7	
3.04–3.81	Trace grey seams, stiff.	93.68		SS	5	100	15		11.8	
3.81–5.33	Grey, trace wet seams.	92.16		SS	6	100	11		10.9	
				SS	7	100	12		10.8	
5.33–6.09	Firm.	91.40		SS	8	60	6		10.3	
6.09–6.70	Stiff.	90.79		SS	9	50	10		10.1	
6.70	End of Borehole									



Contractor: Strata Drilling Group **Grade Elevation:** 97.49 masl

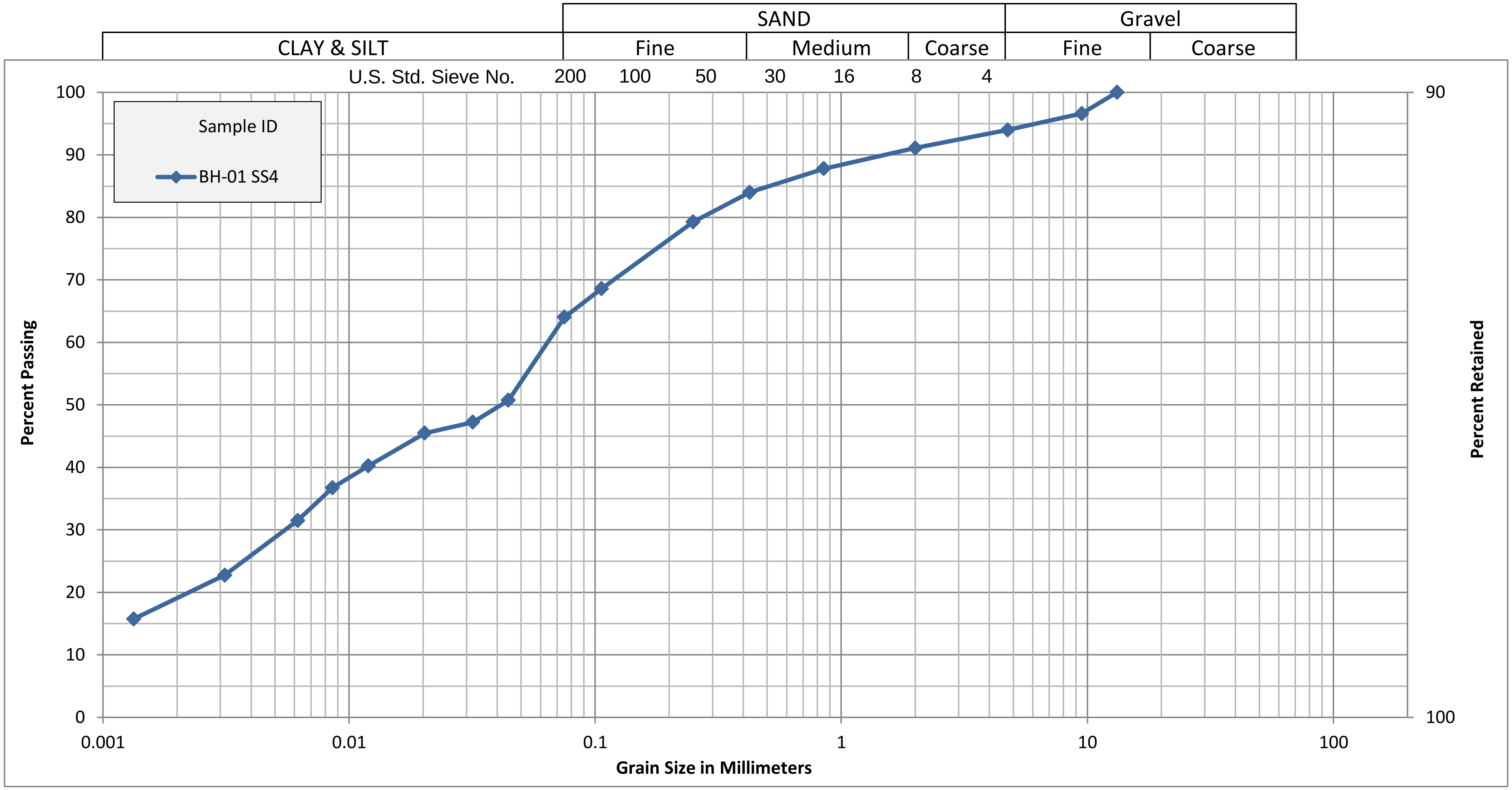
Drilling Method: Direct Push **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

APPENDIX III

Laboratory Testing Reports for Soil Samples

Unified Soil Classification System



Sample ID	Depth (ft)	% Gravel	% Sand	% Silt	% Clay
BH-01 SS4	7.5-9.5	6.0	30.0	46.0	18.0

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Unit 1, Waterloo, Ontario N2K 4M8

PARTICLE SIZE DISTRIBUTION ANALYSIS

Geotechnical Investigation - 1726 Baseline Rd W, Bowmanville, ON
Gay Company Limited

Figure No. 1

361920.000

Reviewed By:

More information available upon request

Atterberg Limits

LS 703&704 / ASTM D4318

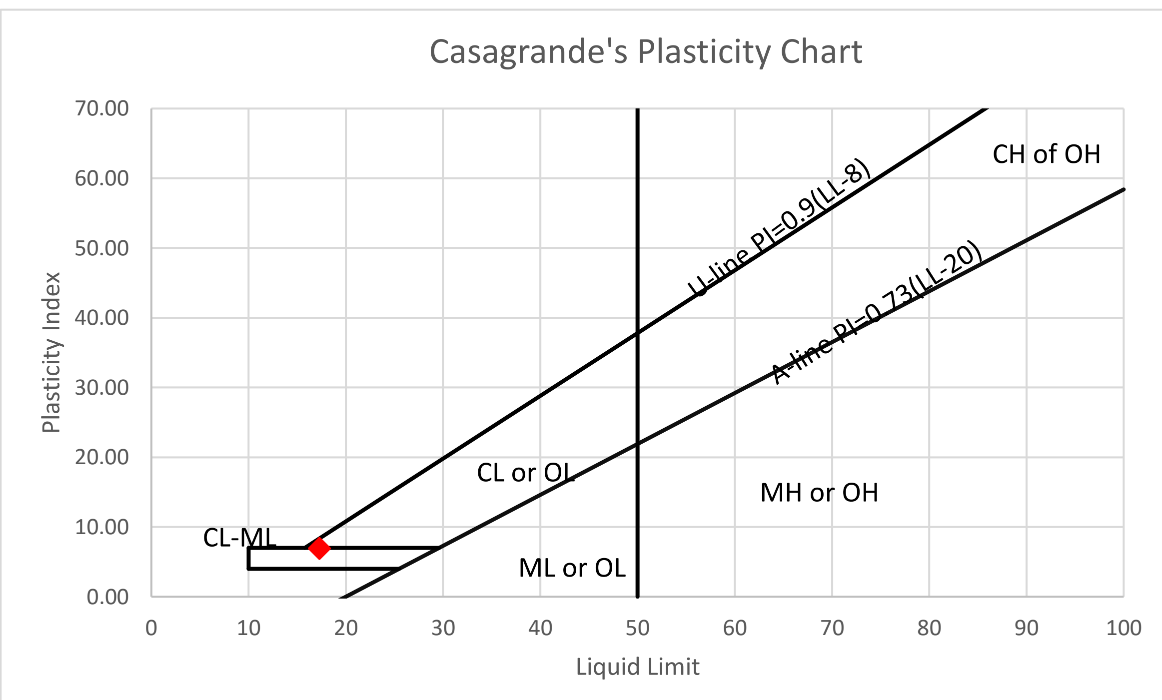
Project Name: Geotechnical Investigation
Project No. 361920.000
Client: Gay Company Limited
Location: 1726 Baseline Rd W, Bowmanville, ON
Material: Soil
Sample: BH-01 SS4 7.5-9.5'

Test Date: October 14, 2025
Tested By: B Frank
Sample Date: September 24, 2025
Sampled By: C Goss
Reviewed By: V Marshall

Liquid Limit - Method A - Mechanical						
Pot Number	1	2	3	4		
Number of blows	29	24	20	15		
Wet mass + pot	32.39	31.40	30.94	30.31		
Dry mass + pot	29.96	29.06	28.56	28.04		
Tare	15.53	15.62	15.35	15.7		
Water content %	16.84	17.41	18.02	18.38		

Plastic Limit - Hand Rolled			
Pot Number	1	2	
Wet mass + pot	24.35	26.80	
Dry mass + pot	23.55	25.75	
Tare	15.91	15.67	
Water content %	10.5	10.4	

PI = LL - PL	
Liquid Limit %	**17**
Plastic Limit %	**10**
Plastic Index	**7**
Non Plastic	



** More information available upon request*

APPENDIX IV

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report was prepared for the exclusive use of the Client and their authorized agents, subject to the conditions and limitations contained within the duly authorized work plan. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications.

Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any ‘other’ activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor’s responsibility that the appropriate Provincial Health and Safety Act is adhered to, and Site conditions satisfy all ‘other’ acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project.

The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the *Limitations Act*, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.

Document1

Template: Appendix – Report Limitations Guidelines for Use, GEO, January 21, 2025