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Environmental Engineering

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Noise and Vibration Studies

Geotechnical Investigation

Proposed Warehouse Development
Boundary Road at Thunder Road
Ottawa, Ontario

Prepared For

Avenue 31

July 22, 2021

Report PG5161-1 Revision 3

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Avenue 31 to conduct a geotechnical investigation for the proposed warehouse development to be located at Boundary Road and Thunder Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the investigation were to:

☐ Determine the subsoil and groundwater conditions at this site by means of boreholes.

☐ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

2.0 Proposed Development

Based on the preliminary plans, it is understood that the proposed development will consist of a commercial warehouses facilities. The proposed buildings will be of slab-on-grade construction. Parking areas, loading docks and associated driveways connecting to both Thunder Road and Boundary Road are expected. Truck traffic will be a large component of the vehicle loading on the pavement structure.

3.0 Method of Investigation

3.1 Field Investigation

Field Programs

Prior to undertaking this new assignment, existing geotechnical information was available from a previous environmental investigation carried out by Paterson for the subject site on December 19, 2018. At that time, a total of 3 boreholes were drilled to a maximum depth of 4.2 m to assess the subsurface soil conditions.

An investigation was carried out on June 30 and July 2, 2020. At that time a total of 7 boreholes were drilled to a maximum depth of 7.5 m to assess the subsurface soil conditions.

A supplemetnal investigation was carried out on April 15, 2021 along the existing residential area on the north portion of the site and on July 14, 2021 along the central treed portion of the site. At the time a total of 6 boreholes were drilled to a maximum depth of 5.8 m.

The test hole locations are shown on the enclosed drawing PG5161-1 - Test Hole Location Plan.

The boreholes were completed with a track-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer. The borehole procedure consisted of augering, or advancing a casing by rotary drilling, to the required depths at the selected locations, and sampling and testing the overburden soils.

Sampling and In Situ Testing

Soil samples were recovered using a split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of each of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength testing, using a vane apparatus, was carried out at regular intervals of depth in cohesive soils.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at 2 borehole locations. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are presented on the Soil Profile and Test Data sheets in Appendix 1 of this report.

Groundwater

Boreholes of the previous investigation were outfitted with 51 mm water monitoring well. Flexible standpipe piezometers were installed in all other boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in subsection 4.3 and noted on the Soil Profile and Test Data sheets presented in Appendix 1.

Sample Storage

All samples from the supplemental geotechnical investigation will be stored in the laboratory for a period of one month after issuance of this report. They will then be discarded unless we are directed otherwise.

3.2 Field Survey

The test hole locations were selected by Paterson personnel in a manner to provide general coverage of the proposed development, taking into consideration site features.

The borehole locations and ground surface elevations completed for our previous environmental investigation were surveyed by Annis, O'Sullivan, Vollebekk Ltd. The current investigation borehole locations and ground surface elevations were surveyed by Paterson personnel and reference a geodetic datum (NAD83). Both are presented on Drawing PG5161-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging. A total of 7 samples were submitted to Atterberg Limits testing and sieves and/or hydrometer analysis was completed on 2 representative samples.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

One soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. The results are presented in Appendix 1 and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is undeveloped and trees have been recently cleared on the south portion of the site. The north western portion of the site consists of a mature treed area. The site is bordered by Thunder Road to the northeast, residential dwellings and wooden area to the northwest, Boundary Road and commercial properties to the east, and treed land to the south and west. The existing ground surface is relatively flat and range across from an elevations of approximately 76 to 78 m. Excavated drainage ditches were also encountered at the subject site. Wet ground and surface water was encountered along the south and southwestern property borders.

The north portion of the site is separated by a creek. At the time of the investigation the creek was noted to be blocked by a beaver dam. The water was backflowing through a series of ditches located on the south portion of the site and towards the treed area. The ditches were noted to be full with 0.6 to 1.2 m of water.

4.2 Subsurface Profile

Overburden

The subsurface profile encountered at the test hole locations generally consists of topsoil and/or organic material extending to approximate depths of 100 to 250 mm below the existing ground surface. A brown silty sand, trace clay was generally encountered underlying the topsoil, extending to depths of 0.7 to 1.3 m below ground surface. A firm, brown to grey silty clay deposit with sand seams was observed underlying the silty sand to sand layer. Practical refusal to the DCPT was encountered at a depth of 16 to 21.1 m.

It should be noted that a fill layer varying in thickness from 0.6 to 0.75 m was encountered around the existing residential dwelling on the north portion of the site.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Bedrock

Based on available geological mapping, the bedrock in this area consists of shale of the Carlsbad formation with an overburden drift thickness of 25 to 35 m.

4.3 Groundwater

Groundwater levels were measured in the piezometers at the boreholes BH 1-20 through BH 7-20, as well as in the monitoring wells from the previous investigation (BH 1 and BH 2) on January July 22, 2020. The measured groundwater level (GWL) readings are presented in Table 1 below. Based on our field observations, experience with the local area, moisture levels and the colouring of the recovered samples, it is expected that the groundwater level is between 0.5 to 2 m below the existing grade. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore groundwater levels could differ at the time of construction.

Table 1 - Summary of Groundwater Levels

Borehole Number	Ground Surface Elev. (m)	Measured Groundwater Level		Recording Date
		Depth (m)	Elevation (m)	
BH 1-21	76.96	1.25	75.71	April 28, 2021
BH 2-21	76.76	0.05	76.71	April 28, 2021
BH 3-21	76.33	0.91	75.42	July 21, 2021
BH 4-21	76.53	Blocked	Blocked	July 21, 2021
BH 5-21	76.34	0.73	75.61	July 21, 2021
BH 6-21	76.30	0.78	75.52	July 21, 2021
BH 1-20	76.32	5.87	70.45	July 22, 2020
BH 2-20	76.62	0.70	75.92	July 22, 2020
BH 3-20	76.90	0.98	75.92	July 22, 2020
BH 4-20	76.46	3.12	73.34	July 22, 2020
BH 5-20	77.03	2.23	74.80	July 22, 2020
BH 6-20	76.93	3.09	73.84	July 22, 2020
BH 7-20	76.90	1.15	75.75	July 22, 2020
BH 1	77.10	1.49	75.61	July 22, 2020
BH 2	76.82	0.92	75.90	July 22, 2020

Note: Ground surface elevations at the test hole locations were recorded by Paterson Personnel and are referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered satisfactory for the proposed warehouse development. Conventional shallow foundations are expected for the proposed buildings provided that the design loads can be achieved based on the bearing resistance values provided. For buildings where design loads exceed the bearing resistance values, then end bearing piles will be required to handle the design building loads. End bearing pile capacities and uplift resistance values have been provided in Subsection 5.3. Also, bearing capacities for conventional shallow footings have been provided in Subsection 5.3 for any lightweight structures to be constructed at the subject site.

Due to the presence of the deep silty clay deposit, a permissible grade raise restriction will be applied for the subject site.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, containing deleterious (debris and unusable fill) or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures.

Fill Placement

Fill used for grading beneath the proposed building should be in accordance with the recommendations provided in Subsection 5.6 - Slab-on-Grade below. These materials should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill, along with site-excavated soil, can be used as general landscaping fill where settlement of the ground surface is of minor concern, in accordance with the permissible grade raise recommendations provided in Subsection 5.4. This material should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless used in conjunction with a composite drainage membrane.

5.3 Foundation Design

Shallow Foundations - Bearing Resistance Values

Pad footings, up to 9 m wide, and strip footings, up to 6 m wide, placed on an undisturbed, stiff silty clay bearing surface can be designed using a bearing resistance value at SLS of **60 kPa** and a factored bearing resistance value at ULS of **90 kPa.**

Footings placed on an undisturbed, silty sand bearing surface with a minimum separation of 500 mm with the silty clay layer can be designed using a bearing resistance value at SLS of **80 kPa** and a factored bearing resistance value at ULS of **125 kPa.**

The bearing resistance values are provided on the assumption that the footings will be placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in-situ or not, have been removed, prior to the placement of concrete for footings.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to the in-situ bearing medium soils above the groundwater table when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in-situ soil of the same or higher capacity as the bearing medium soil.

Settlement

The total and differential settlements will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

The potential post construction total and differential settlements are dependent on the position of the long term groundwater level when buildings are situated over deposits of compressible silty clay. Efforts can be made to reduce the impacts of the proposed development on the long term groundwater level by placing clay dykes in the service trenches, reducing the sizes of paved areas, leaving green spaces to allow for groundwater recharge or limiting planting of trees to areas away from the buildings. However, it is not economically possible to control the groundwater level.

End Bearing Piled Foundation

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance values at ultimate limit states (ULS) are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored at ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance	Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		Factored at ULS (kN)		
245	9	1495	25	40
245	11	1750	24	48.5
245	13	2000	25	56

The minimum centre-to-centre pile spacing is 2.5 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Due to the proposed grade raises at the site, downdrag loads should be considered on the piles. Based on the available subsurface information, it is expected that the piles will be driven through stiff to soft silty clay. The silty clay generally has a cohesion of 20 to 40 kPa. Assigning an adhesion factor of 1.0 to 0.5, the silty clay can be taken to have an ultimate adhesion of 20 kPa against the sides of the piles.

The downdrag load is effectively applied to each pile at the location of the "neutral plane," where negative (i.e. downdrag) skin friction becomes positive shaft resistance. In the case of the end-bearing piles at this site, the neutral plane will be located near the bedrock surface.

The downdrag load is a structural pile capacity criterion and does not affect the geotechnical capacity of the piles. The structural axial capacity of the pile is governed by its structural strength at the neutral plane when subjected to the permanent load plus the downdrag load. Transient live load is not to be included. At or below the pile cap, the structural strength of the embedded pile is determined as a short column subjected to the permanent load plus the transient live load, but downdrag load is to be excluded.

At the depth of the neutral plane where the downdrag load is applied, the pile structure is well confined. The 4th edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles at the neutral plane, for resisting permanent load plus the downdrag load, can be determined by applying a factor of safety of 1.5 to the pile material strength (steel yield and concrete 28 day compressive strength).

Lateral Load Resistance

Lateral loads on the foundations can be resisted using passive resistance on the sides of the foundations. For Limit States Design, the resistance factor to be applied to the ultimate lateral resistance, including passive pressure, is 0.50. The total lateral resistance will be comprised of the individual contributions from up to several material layers, as follows.

Geotechnical parameters for the native sand and for typical backfill materials compacted to 98% of SPMDD in 300 mm lift thicknesses are provided in Table 3, below, along with the associated earth pressure coefficients for horizontal resistance calculations for footings under lateral loads or deadman anchors. Friction factors between concrete and the various subgrade materials are also provided in Table 3, where normal loads allow them to be used.

Where granular soils and/or granular backfill materials are present, the passive pressure can be calculated using a triangular distribution equal to $K_P \cdot \gamma \cdot H$ where:

K_P = factored passive earth pressure coefficient of the applicable retained soil, 1.5
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the equivalent wall or footing side (m)

Note that for cases where the depth to the top of the structure (i.e. footing) pushing against the soil does not exceed 50% of the depth to the base of the structure, the effective value of H in the above noted relationship will be the overall depth to the base of the structure. There will also be “edge effects” where the effective width of soil providing the resistance can be increased by 50% of the effective depth on each side of the pushing structural component.

Note that where the foundation extends below the groundwater level, the effective unit weight should be utilized for the saturated portion of the soil or fill.

Where a component of lateral resistance is to be provided by the EPS foam lightweight fill (LWF) layer, the ultimate passive or lateral resistance will be the compressive strength of the LWF at 5% deformation. A geotechnical resistance factor of 0.5 also applies to this resistance component. In Subsection 5.6 below, the LWF under the slab is recommended to consist of EPS Blocks Type 12, which has a compressive strength at 5% deformation of 35 kPa.

Should additional passive resistance be require, the horizontal component of the axial resistance of battered piles (up to 1H:3V inclination), or anchors can be used in the building foundation design.

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles. Unit weights of materials are provided in Table 3.

For soil above the groundwater level, calculate using the “drained” unit weight and below groundwater level use the “effective” unit weight. Backfilled excavations in low permeability soils can be expected to fill with water and the use of the effective unit weights would be prudent if drainage of the anchor footings is not provided.

As noted, the piles will generally be located below the groundwater level, so the submerged, or effective, weight of the pile will be available to contribute to the uplift resistance, if required. Considering that this is a reliable uplift resistance, and is really counteracting a dead load, it is our opinion that a resistance factor of 0.9 is applicable for the ULS weight component.

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated.

Table 3 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle (°) φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A Fill (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B Type I Fill (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
OPSS Granular B Type II Fill (Crushed Stone)	22.5	14.0	40	0.62	0.20	0.33	10.3
Silty Clay	17.0	9.0	30	0.30	0.33	0.50	3.0
In Situ Silty Sand or Site Excavated Silty Sand Fill	18.0	11.2	32	0.48	0.30	0.46	5.6

Notes:
❑ Properties for fill materials are for condition of 98% of standard Proctor maximum dry density.
❑ The earth pressure coefficients provided are for horizontal backfill profile.
❑ Passive pressure coefficients incorporate wall friction of 0.5 φ'.

Loading Dock

The foundation wall at the loading dock, if the loading dock grade is depressed, will act as a retaining wall. Therefore, it should be designed to resist the lateral earth pressure of the fill material on the inside of the foundation wall. The wall should be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to K_a γ H where:

K_a = 0.35 - active earth pressure coefficient if some movement can be tolerated and
0.5 if no movement can be tolerated
γ = 22 kN/m^3, unit weight of the fill
H = height of the retaining wall, m

It should be noted that the fill on the inside of the wall should consist of free draining material such as OPSS Granular Type I or II.

The excavation side slope of the footing/foundation wall excavation should be tapered at 3H:1V or flatter on the pavement side of the loading dock and backfilled with OPSS Granular B Type I or II to minimize frost heaving. The fill material should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the material's standard Proctor dry density. The depressed area should be properly drained to minimize total and differential frost heaving.

5.4 Permissible Grade Raise Recommendations

Permissible grade raise recommendations have been determined for the proposed development based on the consolidation testing results of samples of the silty clay obtained during the geotechnical investigation. Based on our findings, a permissible grade raise of **0.9 m** is recommended for grading within 6 m of the proposed buildings' footprints and a 1.5 m permissible grade raise recommendation for the access lanes, car and truck parking areas.

For design purposes, the total and differential settlements associated with the combination of grade raises and slab loading conditions are estimated to be 25 and 20 mm, respectively. A post-development groundwater lowering of 0.5 m was assumed.

To reduce potential long term liabilities, consideration should be given to provide means to reduce long term groundwater lowering (e.g. clay dykes, restriction on planting around the structures, etc).

If required, LWF should consist of EPS (expanded polystyrene) Type 12 blocks for placement below the building footprint, which allow for raising the grade without adding a significant load to the underlying soils. However, these materials are expensive and, in the case of the EPS, are more difficult to use under the groundwater level, as they are buoyant, and must be protected against potential hydrocarbon spills. Use lightweight fill within the interior of the building to reduce the fill-related loads.

LWF should be covered by a 8 mil polyethylene liner followed by a non-woven geotextile, such as Terrafix 270 R or equivalent, and a biaxial geogrid, such as Geosynthetics Systems TBX2500 or equivalent for areas within the building footprint and under pavement structures, where required.

5.5 Design for Earthquakes

A seismic site response **Class D** should be used for design of the proposed buildings at the subject site according to the OBC 2012. The soils underlying the site are not susceptible to liquefaction.

5.6 Slab on Grade Construction

With the removal of all topsoil and fill, containing deleterious or organic materials, the native soil will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction. It is recommended that the upper 200 mm of sub-slab fill should consist of an OPSS Granular A crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD. Any soft areas should be removed and backfilled with appropriate backfill material. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, or approved granular alternative material are recommended for backfilling below the floor slab.

Modulus of Subgrade Reaction

A modulus of subgrade reaction of **15 MPa/m** can be provided for subfloor cross-section above.

5.7 Pavement Structure

Minimum Pavement Structure Recommendations

Car only parking areas, heavy truck parking areas and access lanes are anticipated at this site. The proposed pavement structures are presented in Tables 4 and 5.

Table 4 - Recommended Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
50	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
300	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or LWF (see below)	

Table 5 - Recommended Pavement Structure
Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	Wear Course - HL-3 or Superpave 12.5 Asphaltic Concrete
50	Binder Course - HL-8 or Superpave 19.0 Asphaltic Concrete
150	BASE - OPSS Granular A Crushed Stone
450	SUBBASE - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil, or OPSS Granular B Type I or II material placed over in situ soil or LWF (see below)	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

LWF (EPS Type 15) blocks, where required below the flexible and rigid pavement structure, should be covered by a 8 mil polyethylene liner, a non-woven geotextile layer, such as Terrafix 270R or equivalent, and a biaxial geogrid, such as Geosynthetics TBX 2500 or equivalent, should be used to separate the granular material from the LWF. The LWF blocks should be placed at least 1 m below the finished grade.

Rigid Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the dolly pads and loading dock should consist of Category C1, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 6 below.

Table 6 - Recommended Rigid Pavement Structure - Loading Dock Exterior Apron

Thickness (mm)	Material Description
200	**Exposure Class C1 - 32 MPa Concrete** (5 to 8% Air Entrainment) with minimum structural reinforcement
100	HI-60 rigid insulation
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over native soil or LWF (see below)	

To control cracking due to shrinking of the concrete slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete apron. The control joints are generally recommended to be spaced at approximately 24 to 36 times the slab thickness (for example; a 200 mm thick slab should have control joints spaced between 4.8 and 7.2 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Pavement Structure Drainage

The pavement structure performance is dependent on the moisture condition at the contact zone between the subgrade material and granular base. Failure to provide adequate drainage under conditions of heavy wheel loading could result in the subgrade fines pumped into the stone subbase voids, thereby reducing the load bearing capacity.

Due to the impervious nature of the subgrade materials consideration should be provided to installing subdrains during the pavement construction. The subdrains should be provided for catch basins and extend at least 3 m in four orthogonal directions. The clear crushed stone surrounding the drainage lines should be wrapped with suitable filter cloth. The subdrain inverts should be approximately 300 mm below subgrade level. The subgrade surface should be shaped to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed building. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage layer connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection for heated structures.

Exterior unheated foundations, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of excavations at the site should be cut back at acceptable slopes from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the excavation to be undertaken by open-cut methods.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. It's expected that during the initial excavation program, once the initial influx of groundwater is addressed, the steady state condition for water infiltration will permit excavation side slopes to remain at 1H:1V.

The subsurface soil is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 95% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the site materials above the cover material if the operations are carried out in dry weather conditions.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material standard Proctor maximum dry density.

For areas where rigid insulation will be used to provide frost protection. It is recommended that the rigid insulation be placed at the pipe obvert to allow for the maximum amount of granular cover over the pipe. Having the insulation at the obvert will provide a more effective insulation detail.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a moderate to very aggressive corrosive environment.

6.8 Tree Planting Restrictions

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for recovered silty clay samples at selected locations throughout the subject site. A shrinkage limit test and sieve analysis testing was also completed on selected soil samples. The shrinkage limit testing indicates a shrinkage limit of 14% with a shrinkage ratio of 1.92. The results of our atterberg limit and sieve testing are presented in Appendix 1.

Based on the results of our testing, the silty clay on site is a low to medium plasticity silty clay (Plasticity index < 40%). **In accordance with the city of Ottawa guidelines, the tree planting setback limits may be reduced to 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided all the following conditions are met:**

☐ The underside of footing (USF) is 2.1 m or greater below the lowest finished grade must be satisfied for footings within 10 m from the tree, as measured from the centre of the tree trunk and verified by means of the Grading Plan as indicated procedural changes below.

- ❑ A small tree must be provided with a minimum of 25 m3 of available soil volume while a medium tree must be provided with a minimum of 30 m3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.

- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect. The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- ❑ Grading surround the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree), as noted on the subdivision Grading Plan.

7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

❏ Review the final grading plan from a geotechnical perspective.

❏ Review of LWF recommendations and design along with the confirmation of its installation.

❏ Observation of all pile installations and review of dynamic monitoring results.

❏ Sampling and testing of the concrete and fill materials used.

❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

❏ Observation of all subgrades prior to backfilling.

❏ Field density tests to determine the level of compaction achieved.

❏ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test hole locations, we request immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Avenue 31 or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
July 22, 2021
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Joey R. Villeneuve, M.A.Sc. P.Eng.

David J. Gilbert, P.Eng.

Report Distribution

☐ Avenue 31
☐ Paterson Group

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTERBERG LIMITS TESTING RESULTS

SIEVE/HYDROMETER ANALYSIS RESULT

ANALYTICAL TESTING RESULTS

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5161**

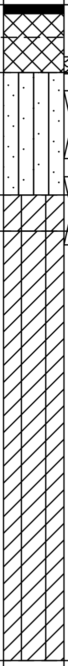
REMARKS

HOLE NO. **BH 1-21**

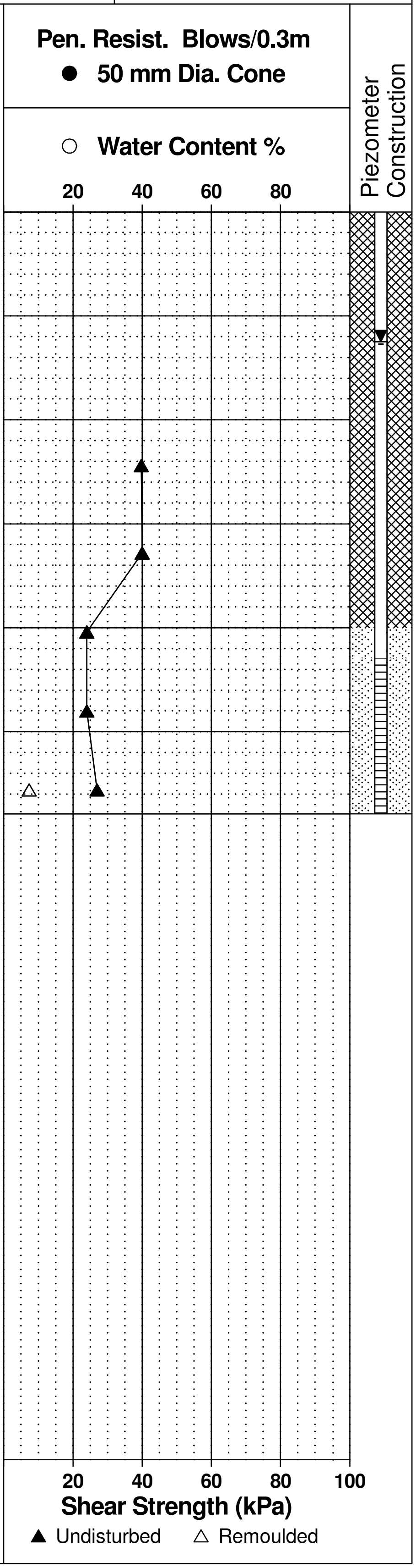
BORINGS BY Track-Mount Power Auger

DATE April 15, 2021

SOIL DESCRIPTION	Depth	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.08				
FILL: Brown silty sand, trace topsoil	0.60	AU	1		
Compact, brown **SILTY SAND**	1.68	SS	2	33	12
Stiff, brown **SILTY CLAY**	2.00	SS	3	25	2
Firm, grey **SILTY CLAY**	5.79				
End of Borehole					
(GWL @ 1.25m - July 21, 2021)					



DEPTH (m)	ELEV. (m)
0	76.96
1	75.96
2	74.96
3	73.96
4	72.96
5	71.96



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

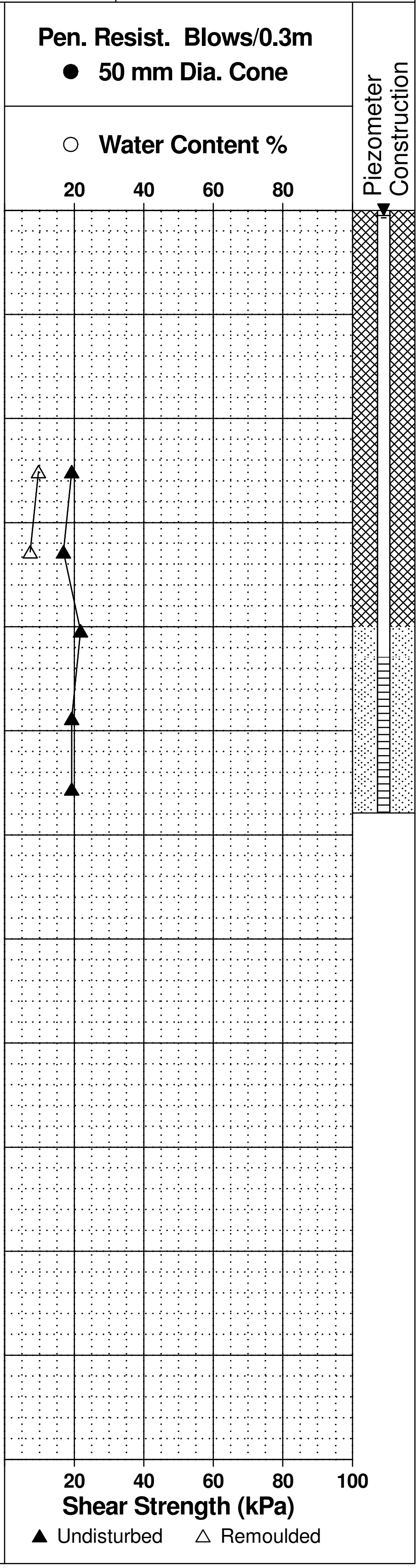
DATE April 15, 2021

FILE NO. PG5161

HOLE NO. BH 2-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.08				
FILL: Brown silty sand with topsoil 0.76	AU	1		
Loose, brown **SILTY SAND** 1.37	SS	2	33	9
Stiff, brown **SILTY CLAY** 2.10	SS	3	50	1
Soft, grey **SILTY CLAY** 5.79	SS	4	100	P
	SS	5	100	P
Dynamic Cone Penetration Test commenced at 5.79m depth. Cone pushed to 19.2m depth.				

DEPTH (m)	ELEV. (m)
0	76.76
1	75.76
2	74.76
3	73.76
4	72.76
5	71.76
6	70.76
7	69.76
8	68.76
9	67.76
10	66.76
11	65.76
12	64.76



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5161

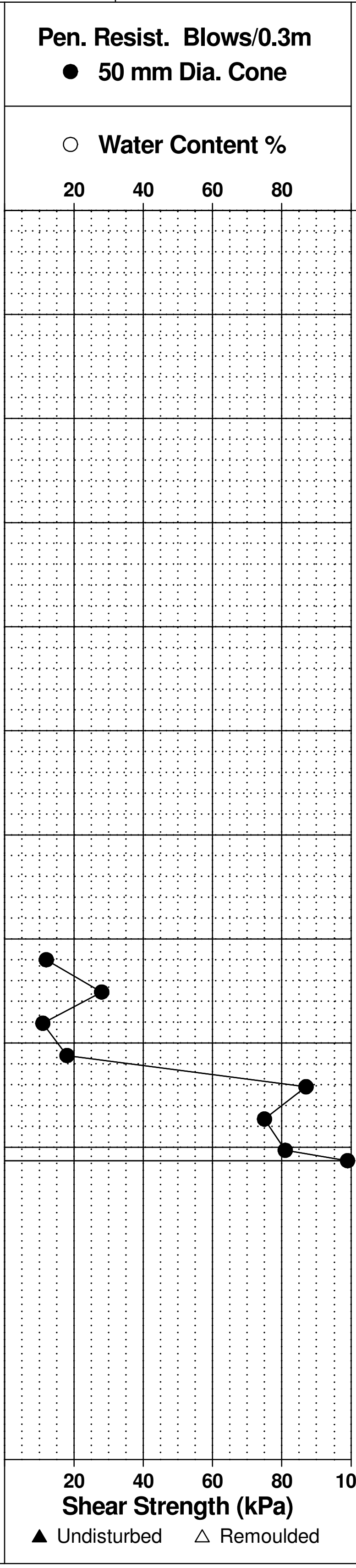
REMARKS

HOLE NO. BH 2-21

BORINGS BY Track-Mount Power Auger

DATE April 15, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE								
						12	64.76	
						13	63.76	
						14	62.76	
						15	61.76	
						16	60.76	
						17	59.76	
						18	58.76	
						19	57.76	
						20	56.76	
						21	55.76	
21.13 End of Borehole Practical DCPT refusal at 21.13m depth (GWL @ 0.05m - July 21, 2021)								



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

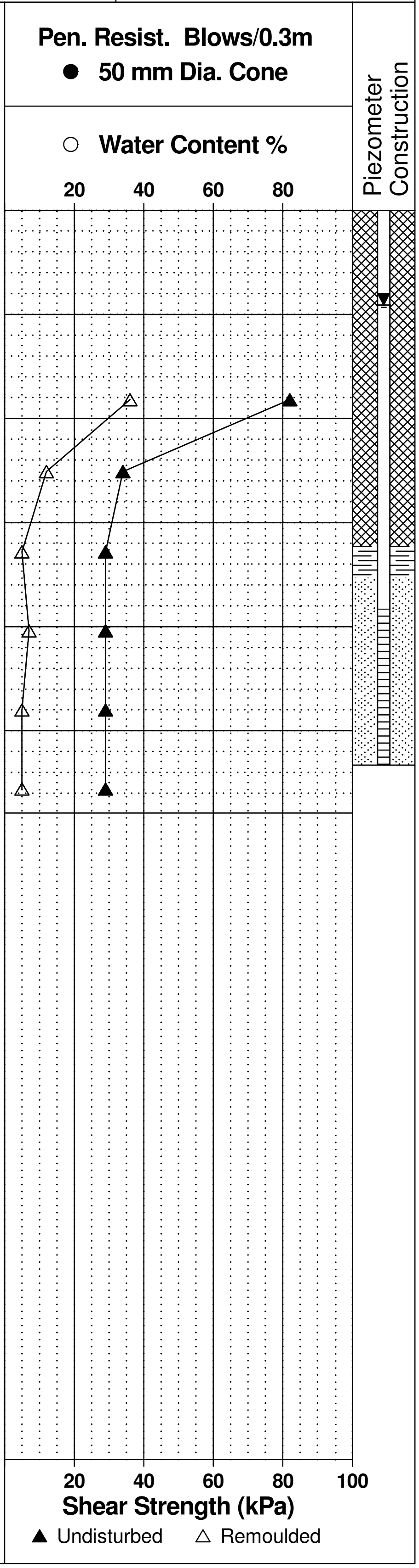
BORINGS BY Track-Mount Power Auger

DATE July 14, 2021

FILE NO. PG5161

HOLE NO. BH 3-21

SOIL DESCRIPTION		SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.33
TOPSOIL	0.25	AU	1				
Compact, brown **SILTY SAND,** trace clay	0.46	AU	2				
Very stiff to stiff, brown **SILTY CLAY** with sand		SS	3	79	4	1	75.33
	1.50						
Firm, grey **SILTY CLAY**		SS	4	75	P	2	74.33
						3	73.33
						4	72.33
						5	71.33
	5.79						
End of Borehole							
(GWL @ 0.91m - July 21, 2021)							



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

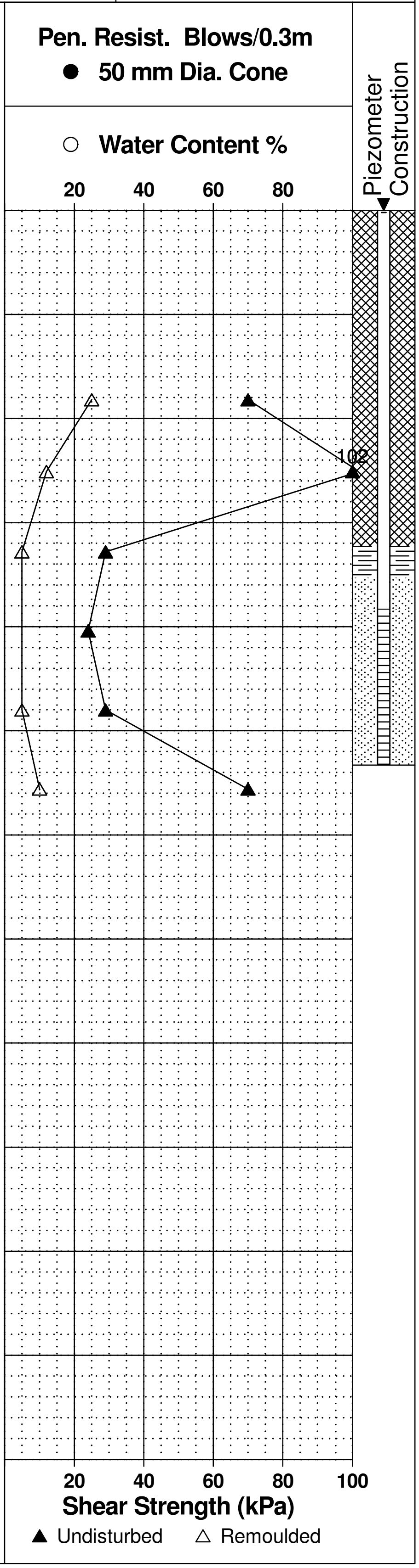
DATE July 13, 2021

FILE NO. PG5161

HOLE NO. BH 4-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.28	AU	1		
Compact, brown **SILTY SAND**	0.76	AU	2		
Stiff to very stiff, brown **SILTY CLAY,** trace sand	3.00	SS	3	83	4
		SS	4	83	P
Firm to stiff, grey **SILTY CLAY**	5.79				
Dynamic Cone Penetration Test commenced at 5.79m depth. Cone pushed to 18.0m depth.					

DEPTH (m)	ELEV. (m)
0	76.53
1	75.53
2	74.53
3	73.53
4	72.53
5	71.53
6	70.53
7	69.53
8	68.53
9	67.53
10	66.53
11	65.53
12	64.53



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5161

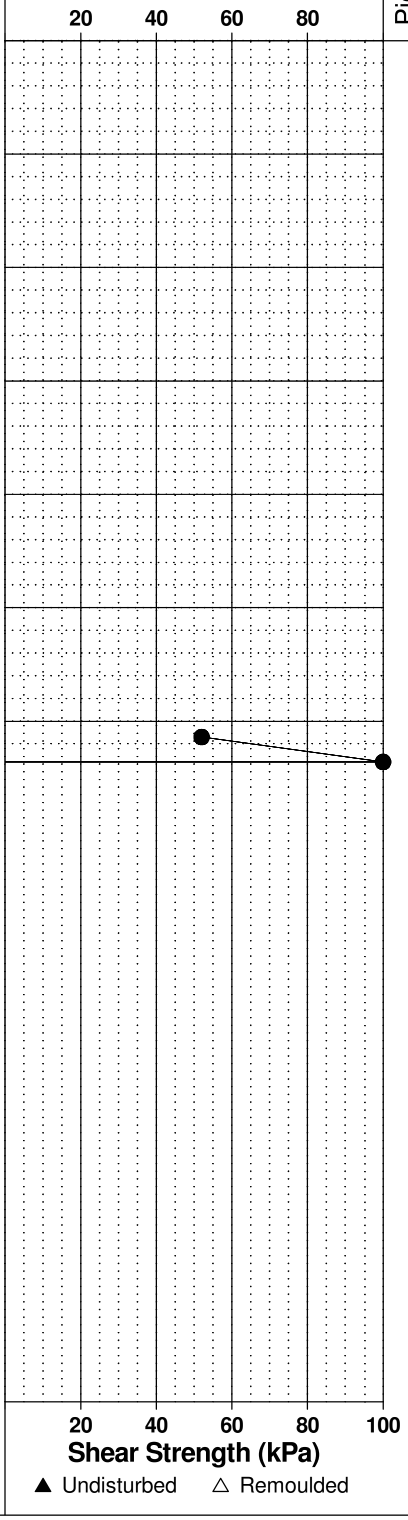
REMARKS

HOLE NO. BH 4-21

BORINGS BY Track-Mount Power Auger

DATE July 13, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
GROUND SURFACE									
						12	64.53		
						13	63.53		
						14	62.53		
						15	61.53		
						16	60.53		
						17	59.53		
						18	58.53		
18.36 End of Borehole Practical DCPT refusal at 18.36m depth									



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 14, 2021

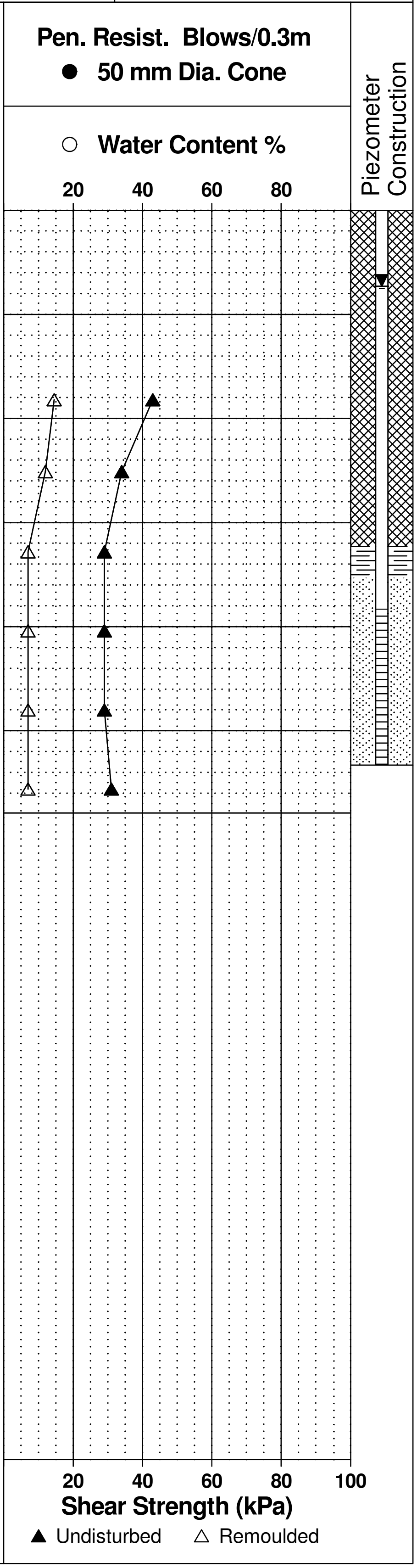
FILE NO. PG5161

HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.34
TOPSOIL 0.25		AU	1				
Compact, brown **SILTY SAND,** trace clay 0.41		AU	2				
		AU	3				
Very stiff to stiff, brown **SILTY CLAY** with sand 1.50		SS	4	67	3	1	75.34
Firm, grey **SILTY CLAY,** trace sand 5.79		SS	5	58	P	2	74.34
						3	73.34
						4	72.34
						5	71.34

End of Borehole

(GWL @ 0.73m - July 21, 2021)



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SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

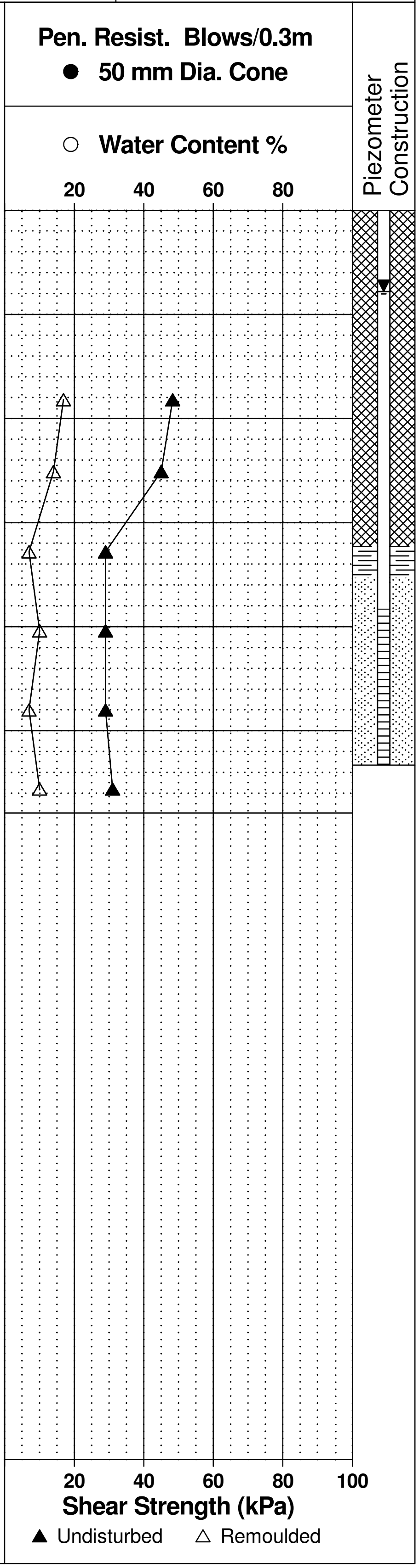
BORINGS BY Track-Mount Power Auger

DATE July 14, 2021

FILE NO. PG5161

HOLE NO. BH 6-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.30
TOPSOIL 0.25		AU	1				
Comapct, brown **SILTY SAND** with topsoil 0.46		AU	2				
Very stiff to stiff, brown **SILTY CLAY** with sand		AU	3				
		SS	4	58	6	1	75.30
1.50							
Firm, grey **SILTY CLAY,** trace sand		SS	5	71	P	2	74.30
						3	73.30
						4	72.30
						5	71.30
5.79							
End of Borehole							
(GWL @ 0.78m - July 21, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

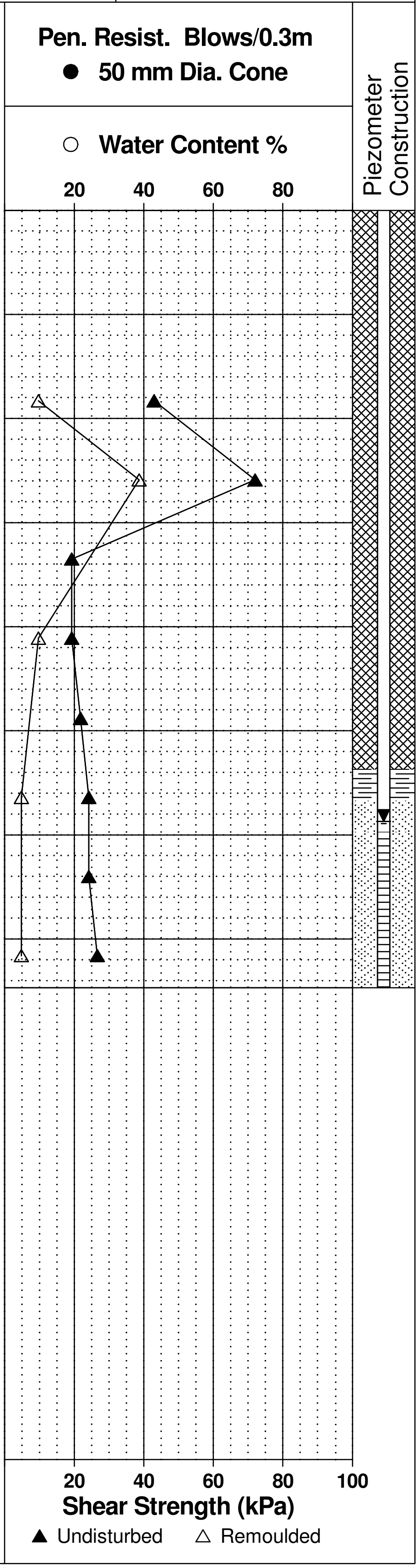
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5161

HOLE NO. BH 1-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.32
TOPSOIL 0.25		AU	1				
Brown **SILTY SAND** 0.38							
		SS	2	83	3	1	75.32
Firm, brown **SILTY SAND**						2	74.32
- soft to firm and grey by 3.0m depth						3	73.32
						4	72.32
						5	71.32
						6	70.32
						7	69.32
7.47							
End of Borehole (GWL @ 5.87m - July 22, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

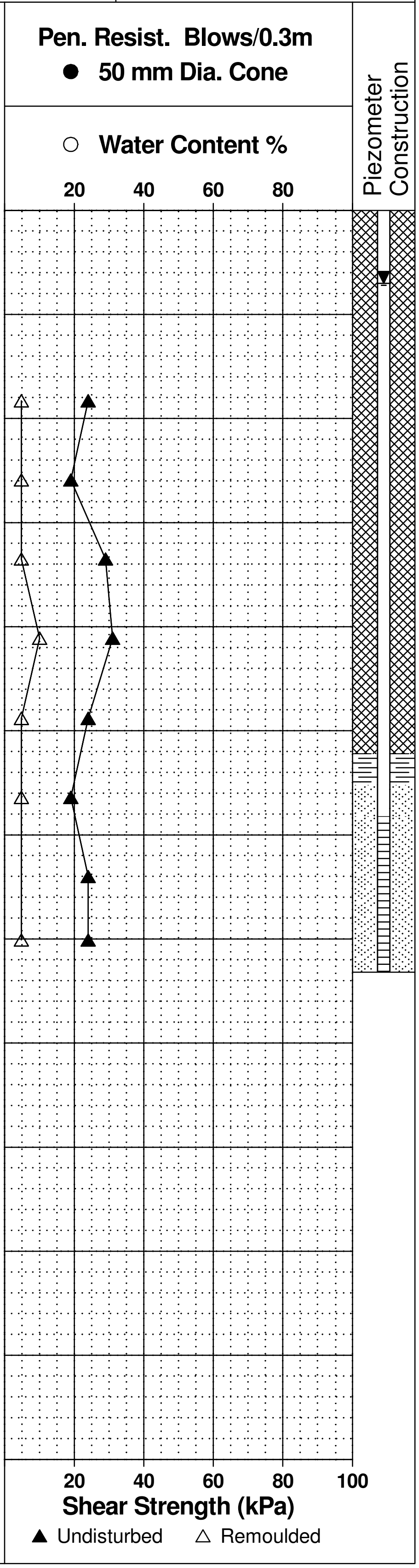
DATE July 1, 2020

FILE NO. PG5161

HOLE NO. BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
Very loose, brown **SILTY SAND,** some organics 0.56		AU	1		
Brown **SILTY SAND** with sand seams 1.52		SS	2	79	3
Firm to soft, grey **SILTY CLAY** 7.32					
Dynamic Cone Penetration Test commenced at 7.32m depth. Cone pushed to 19.5m depth.					

DEPTH (m)	ELEV. (m)
0	76.62
1	75.62
2	74.62
3	73.62
4	72.62
5	71.62
6	70.62
7	69.62
8	68.62
9	67.62
10	66.62
11	65.62
12	64.62



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE July 1, 2020

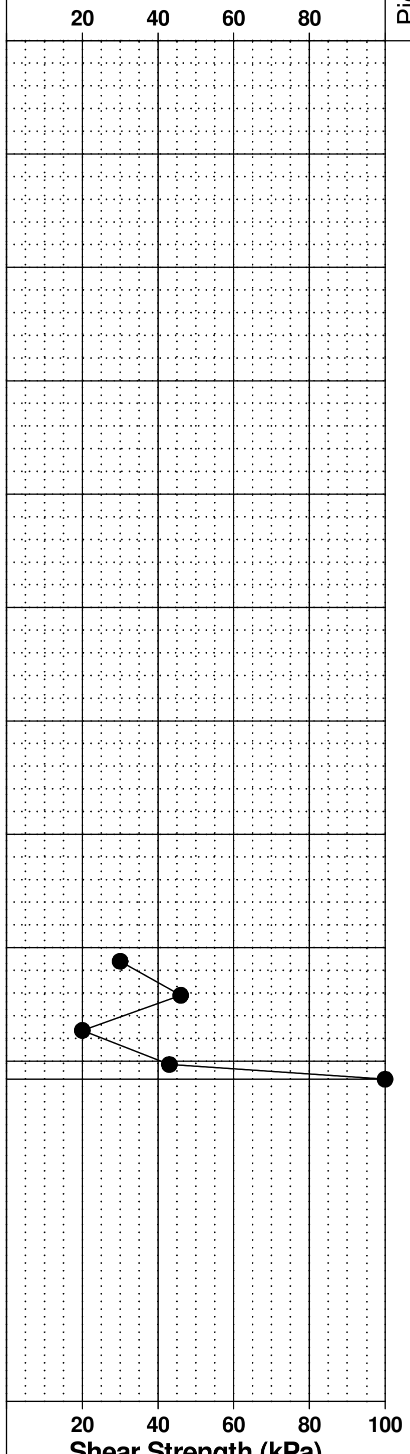
FILE NO. PG5161

HOLE NO. BH 2-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						12	64.62
						13	63.62
						14	62.62
						15	61.62
						16	60.62
						17	59.62
						18	58.62
						19	57.62
						20	56.62
21.16						21	55.62

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %

Piezometer Construction



Shear Strength (kPa)
▲ Undisturbed △ Remoulded

End of Borehole

Practical DCPT refusal at 21.16m depth

(GWL @ 0.70m - July 22, 2020)

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

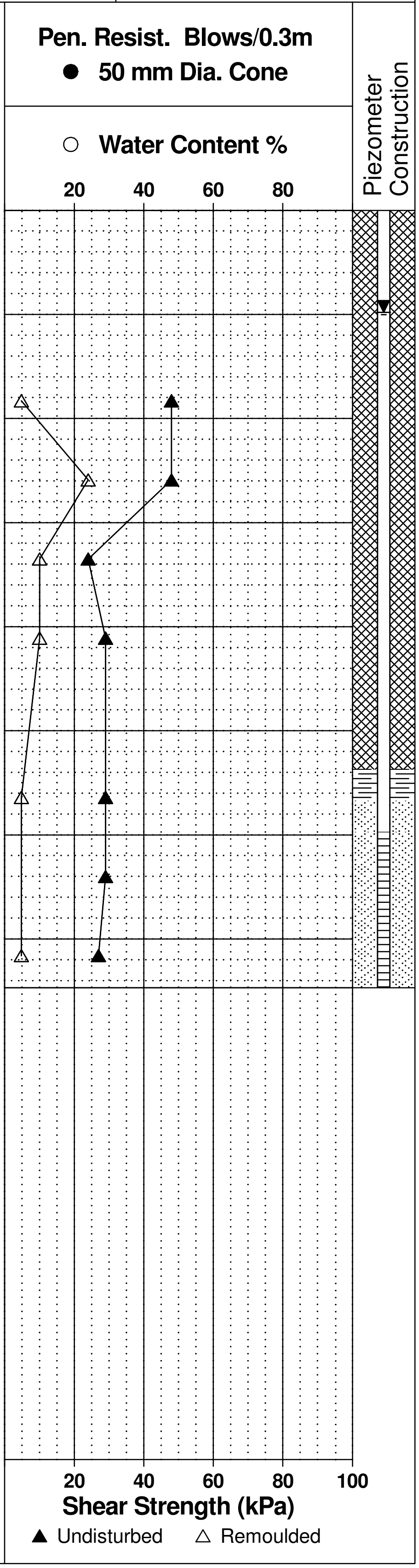
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5161

HOLE NO. BH 3-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.90
Loose, brown **SILTY SAND,** some organics, trace clay		AU	1				
		SS	2	58	8	1	75.90
1.27							
Firm, brow **SILTY CLAY**						2	74.90
						3	73.90
- grey by 3.0m depth						4	72.90
		SS	3	100	1	5	71.90
						6	70.90
						7	69.90
7.47							
End of Borehole							
(GWL @ 0.98m - July 22, 2020)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

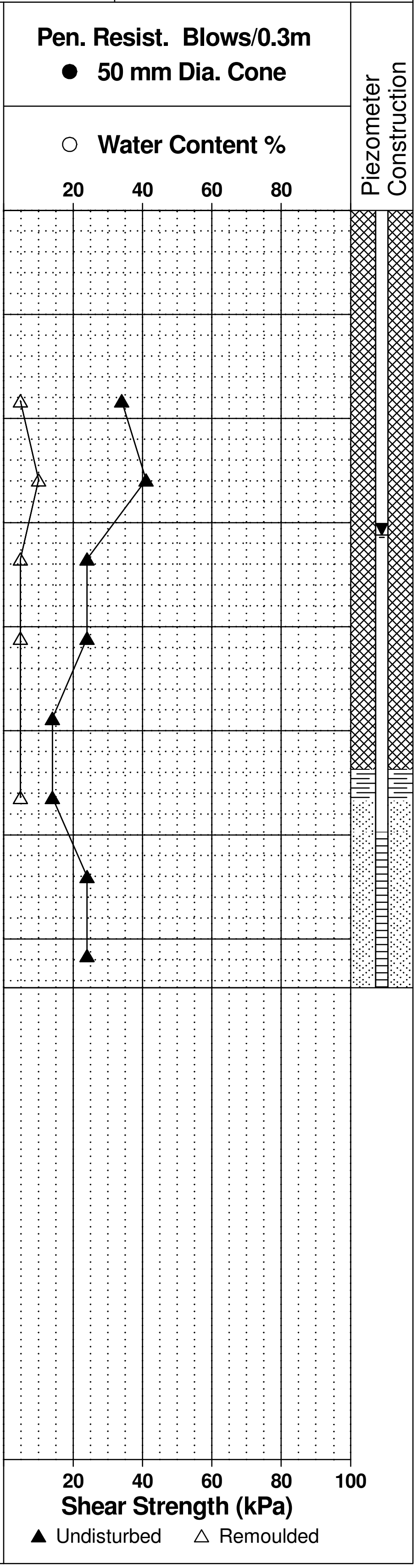
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5161

HOLE NO. BH 4-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.46
TOPSOIL 0.08							
Very loose, brown **SILTY SAND,** trace organics 0.60		AU	1				
		SS	2	46	2	1	75.46
						2	74.46
Firm, brown **SILTY CLAY**							
- soft and grey by 3.0m depth						3	73.46
						4	72.46
						5	71.46
						6	70.46
						7	69.46
7.47							
End of Borehole							
(GWL @ 3.12m - July 22, 2020)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

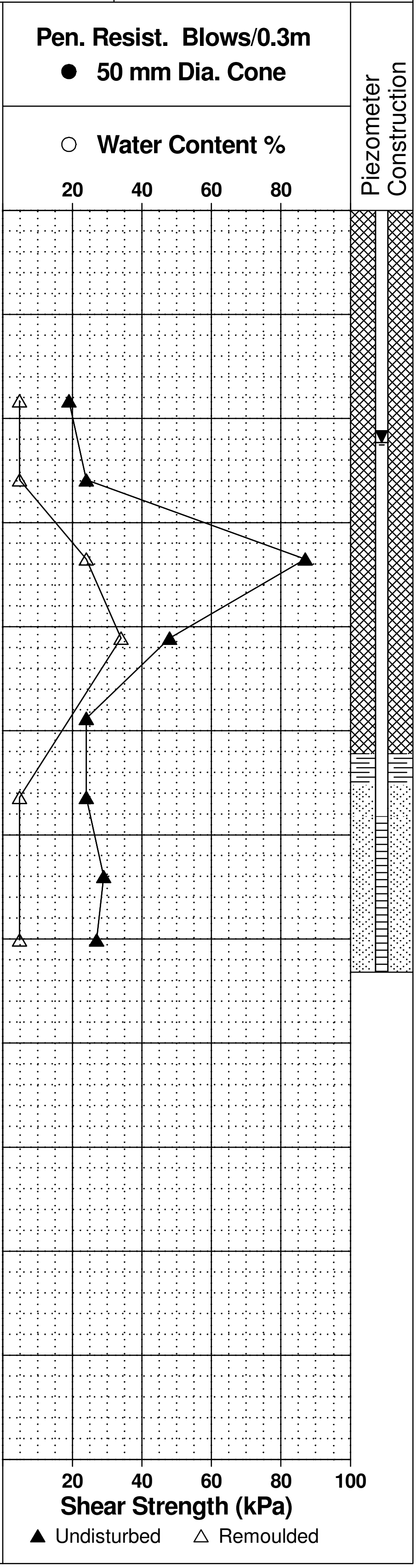
DATE June 30, 2020

FILE NO. PG5161

HOLE NO. BH 5-20

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.10	AU	1		
Loose, brown **SILTY SAND,** trace organics	1.30	SS	2	42	4
Firm, brown **SILTY CLAY**	1.52				
Soft to firm, grey **SILTY CLAY,** trace sand seams	7.32				
Dynamic Cone Penetration Test commenced at 7.32m depth. Cone pushed to 15.2m depth.					

DEPTH (m)	ELEV. (m)
0	77.03
1	76.03
2	75.03
3	74.03
4	73.03
5	72.03
6	71.03
7	70.03
8	69.03
9	68.03
10	67.03
11	66.03
12	65.03



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

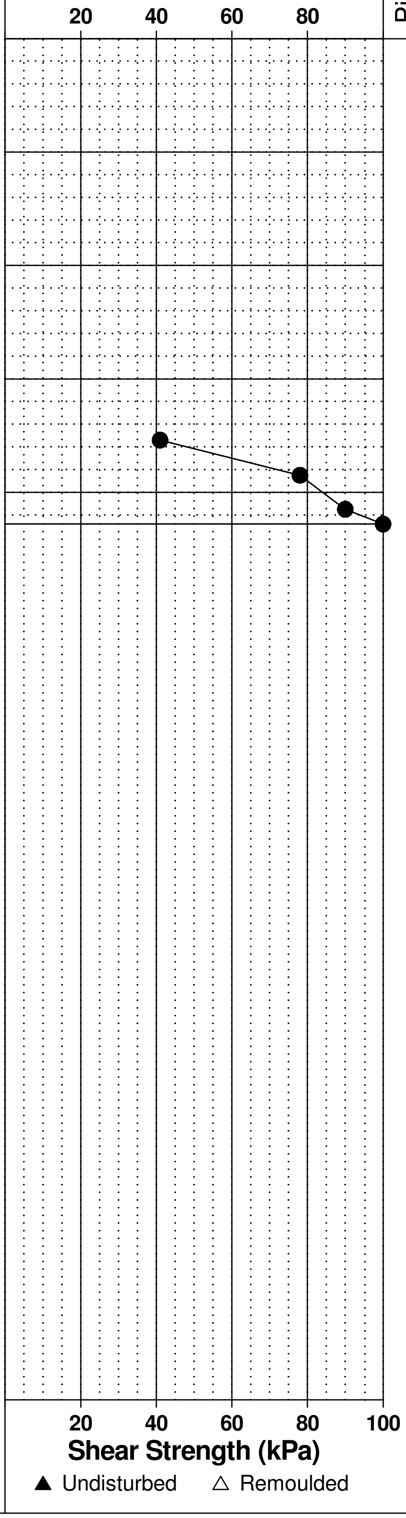
FILE NO. PG5161

HOLE NO. BH 5-20

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						12	65.03
						13	64.03
						14	63.03
						15	62.03
						16	61.03
16.28							
End of Borehole Practical DCPT refusal at 16.28m depth (GWL @ 2.23m - July 22, 2020)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %



Piezometer Construction

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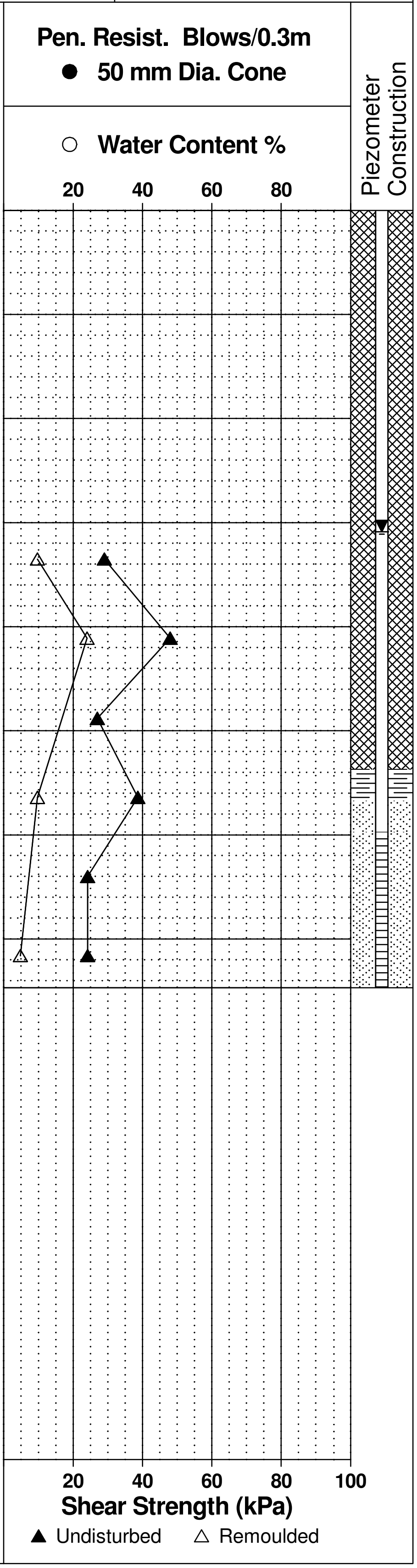
SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Track-Mount Power Auger
DATE June 30, 2020
FILE NO. PG5161
HOLE NO. BH 6-20

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Compact, brown **SILTY SAND**	AU	1		
	SS	2	33	10
1.37				
Firm to soft, grey **SILTY CLAY**	SS	3	100	1
7.47				
End of Borehole				
(GWL @ 3.09m - July 22, 2020)				

DEPTH (m)	ELEV. (m)
0	76.93
1	75.93
2	74.93
3	73.93
4	72.93
5	71.93
6	70.93
7	69.93



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Warehouse Development - Thunder Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

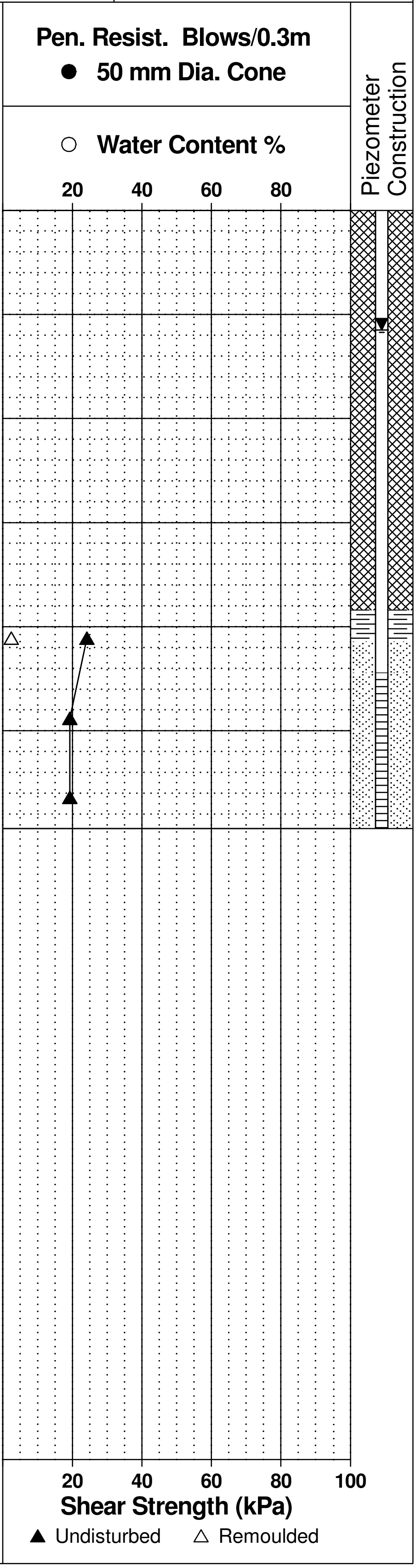
BORINGS BY Track-Mount Power Auger

DATE June 30, 2020

FILE NO. PG5161

HOLE NO. BH 7-20

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	76.90
TOPSOIL 0.15						
FILL: Brown silty sand 0.81	AU	1				
Very loose, brown **SILTY SAND** with clay - grey by 2.3m depth 3.81	SS SS	2 3	100 100	2 W	1 2 3	75.90 74.90 73.90
Soft, grey **SILTY CLAY** 5.94					4 5	72.90 71.90
End of Borehole (GWL @ 1.15m - July 22, 2020)						



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
5368 Boundary Road and 6150 Thunder Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

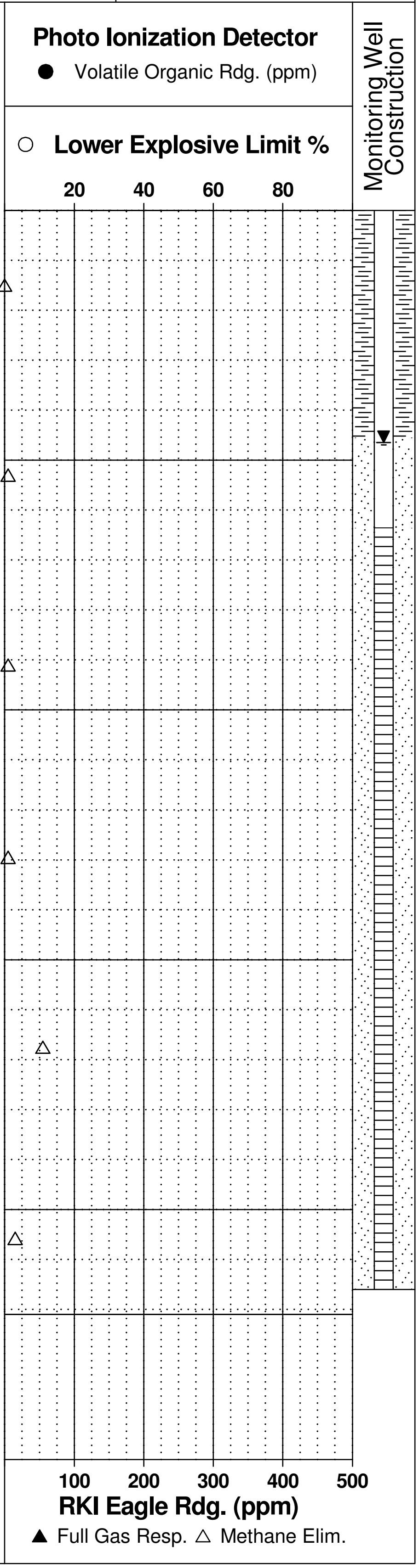
DATE December 19, 2019

FILE NO. PE4480

HOLE NO. BH 1

SOIL DESCRIPTION	DEPTH (m)	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE	0				
TOPSOIL	0.30	AU	1		
Loose, brown **SILTY SAND**	1.52	SS	2	42	6
Brown **SILTY CLAY** - grey by 2.7m depth	4.42	SS	3	83	W
		SS	4	83	W
		SS	5	71	9
		SS	6	96	W
End of Borehole (GWL @ 0.93m - Jan. 14, 2019)					

DEPTH (m)	ELEV. (m)
0	77.22
1	76.22
2	75.22
3	74.22
4	73.22



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
5368 Boundary Road and 6150 Thunder Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 19, 2019

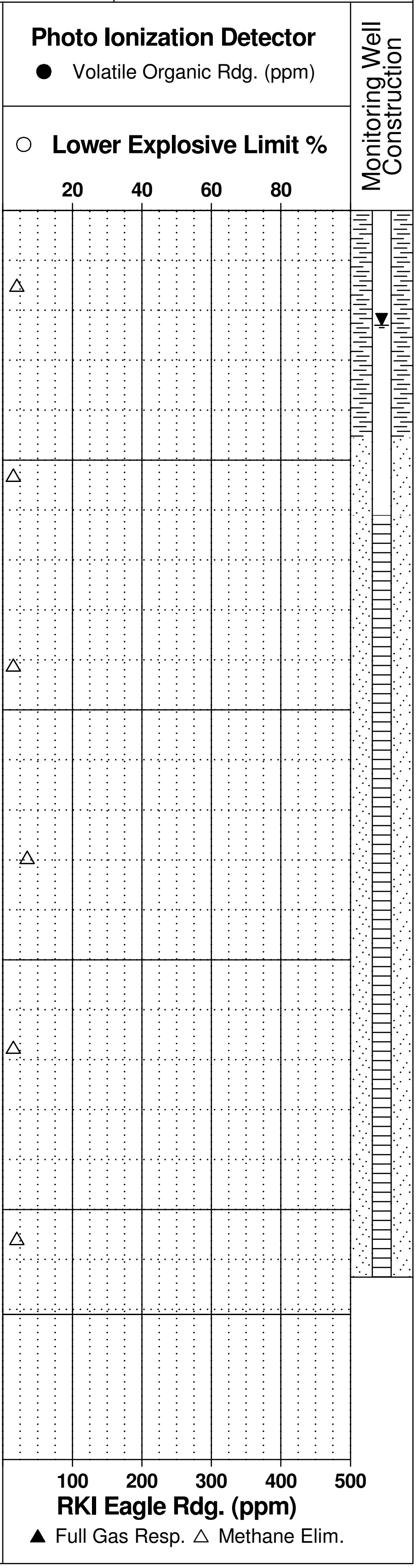
FILE NO. PE4480

HOLE NO. BH 2

SOIL DESCRIPTION	Depth of base (m)
GROUND SURFACE	
TOPSOIL	0.25
Very loose, brown **SILTY SAND**	1.07
Brown **SILTY CLAY** - grey by 2.2m depth	4.42
End of Borehole (GWL @ 0.46m - Jan. 14, 2019)	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	38	2
SS	3	88	W
SS	4	83	4
SS	5	100	W
SS	6	100	W

DEPTH (m)	ELEV. (m)
0	76.76
1	75.76
2	74.76
3	73.76
4	72.76



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SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
5368 Boundary Road and 6150 Thunder Road
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

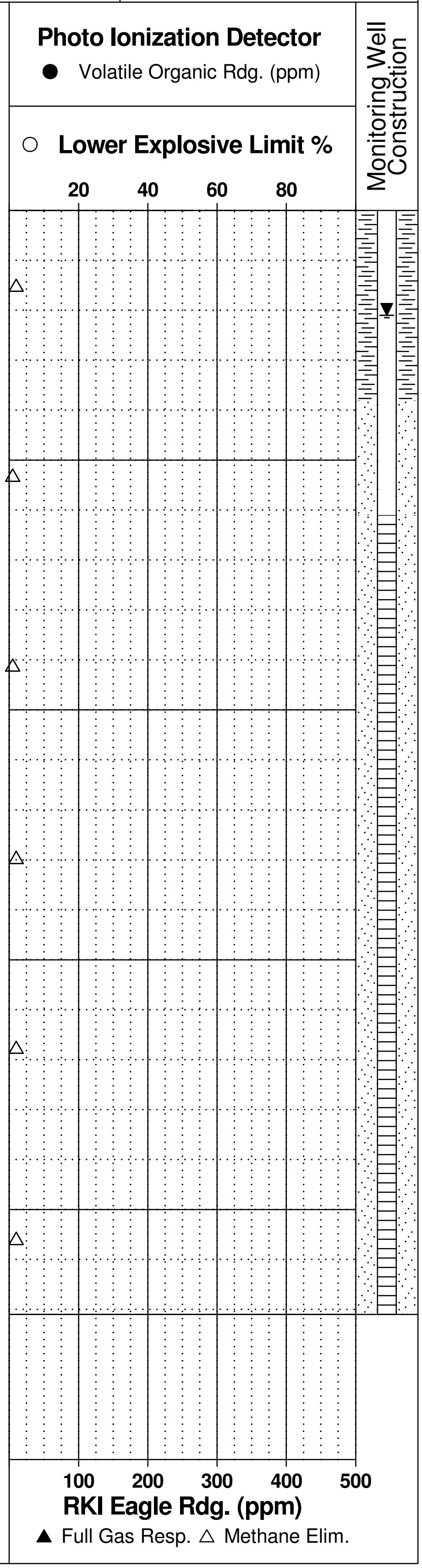
FILE NO. PE4480

REMARKS

HOLE NO. BH 3

BORINGS BY CME 55 Power Auger **DATE** December 19, 2019

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	76.90
TOPSOIL 0.30		AU	1				
Very loose, brown **SILTY SAND** 0.97							
		SS	2	88	3	1	75.90
		SS	3	12	W		
Brown **SILTY CLAY**						2	74.90
		SS	4	100	W		
- grey by 3.0m depth						3	73.90
		SS	5	92	W		
						4	72.90
		SS	6	100	W		
4.42 End of Borehole							
(GWL @ 0.42m - Jan. 14, 2019)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) ‘N’ value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of “P” denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

Compactness Condition	‘N’ Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

Consistency	Undrained Shear Strength (kPa)	‘N’ Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity, S_t, is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity: $S_t < 2$
Medium Sensitivity: $2 < S_t < 4$
Sensitive: $4 < S_t < 8$
Extra Sensitive: $8 < S_t < 16$
Quick Clay: $S_t > 16$

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NQ or larger size core. However, it can be used on smaller core sizes, such as BQ, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	**ROCK QUALITY**
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS - Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW - Thin wall tube or Shelby tube, generally recovered using a piston sampler
G - "Grab" sample from test pit or surface materials
AU - Auger sample or bulk sample
WS - Wash sample
RC - Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC% - Natural water content or water content of sample, %
LL - Liquid Limit, % (water content above which soil behaves as a liquid)
PL - Plastic Limit, % (water content above which soil behaves plastically)
PI - Plasticity Index, % (difference between LL and PL)

Dxx - Grain size at which xx% of the soil, by weight, is of finer grain sizes
These grain size descriptions are not used below 0.075 mm grain size
D10 - Grain size at which 10% of the soil is finer (effective grain size)
D60 - Grain size at which 60% of the soil is finer

Cc - Concavity coefficient = $(D30)^2 / (D10 \times D60)$
Cu - Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

p'_o - Present effective overburden pressure at sample depth
p'_c - Preconsolidation pressure of (maximum past pressure on) sample
Ccr - Recompression index (in effect at pressures below p'_c)
Cc - Compression index (in effect at pressures above p'_c)

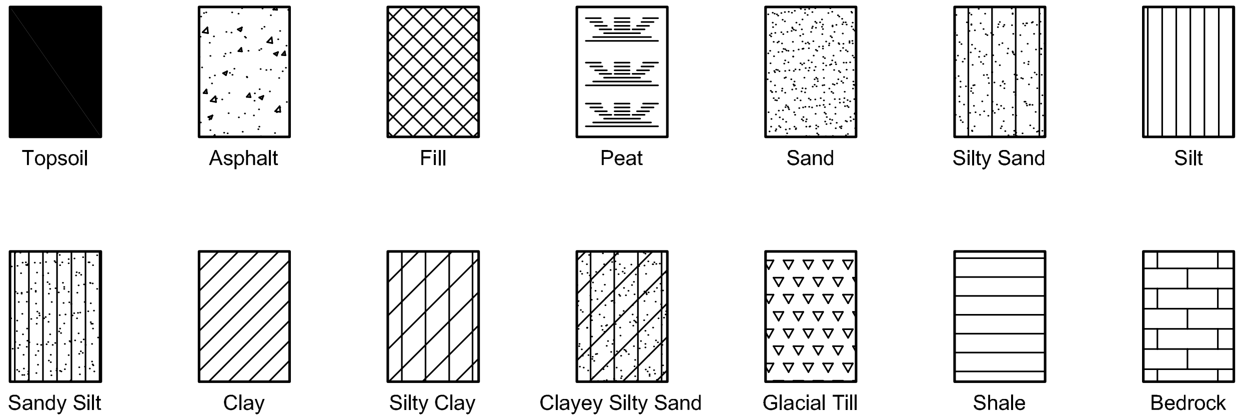
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

k - Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

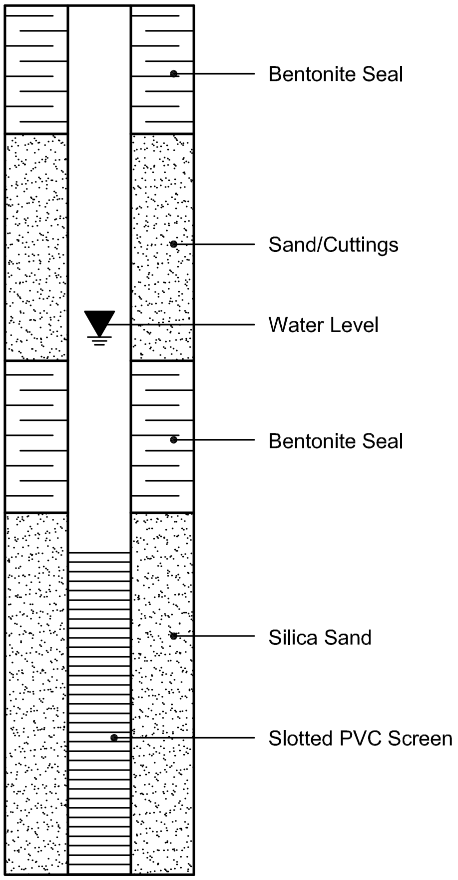
SYMBOLS AND TERMS (continued)

STRATA PLOT

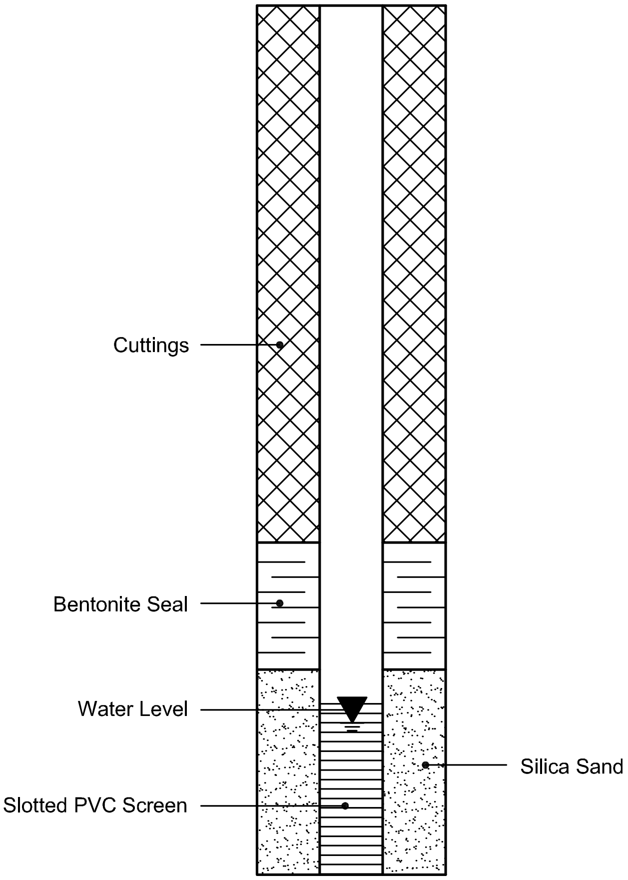


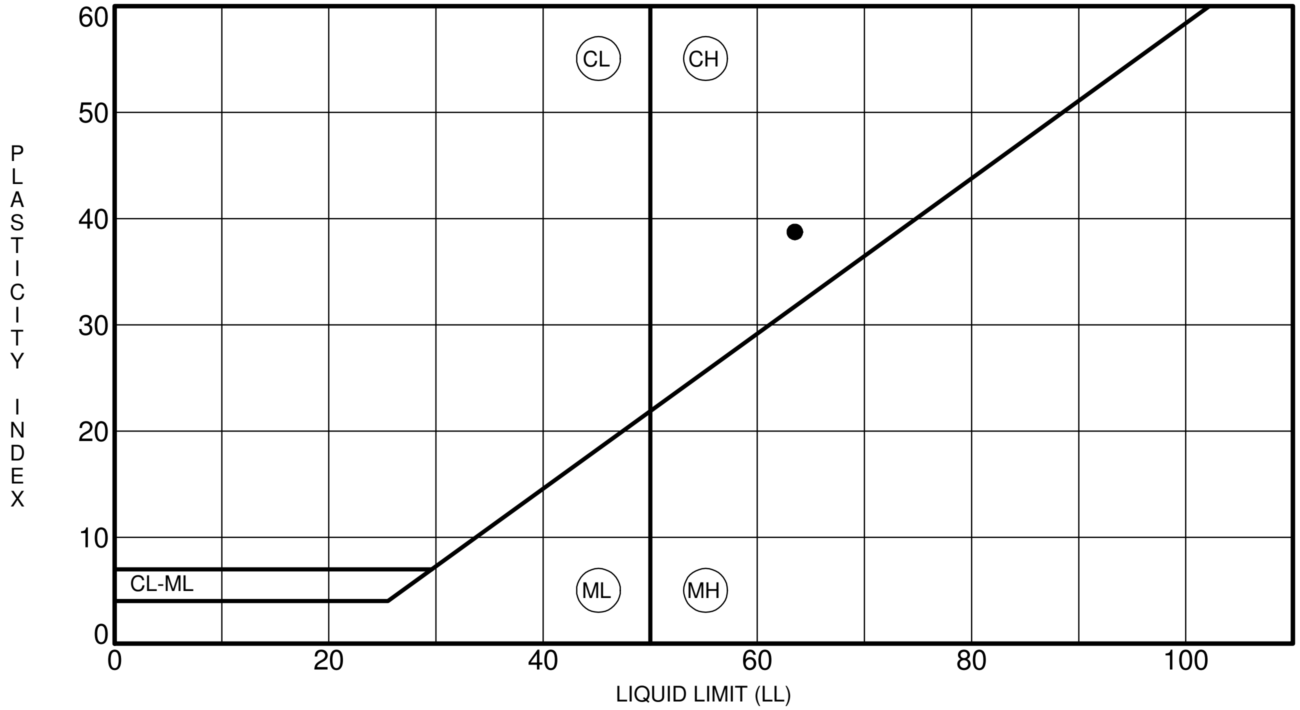
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 2-21 SS 3**	**64**	**25**	**39**		**CH - Inorganic clays of high plasticity**

CLIENT **Exit 96 Developments**

PROJECT **Geotechnical Investigation - Prop. Warehouse Development - Thunder Road**

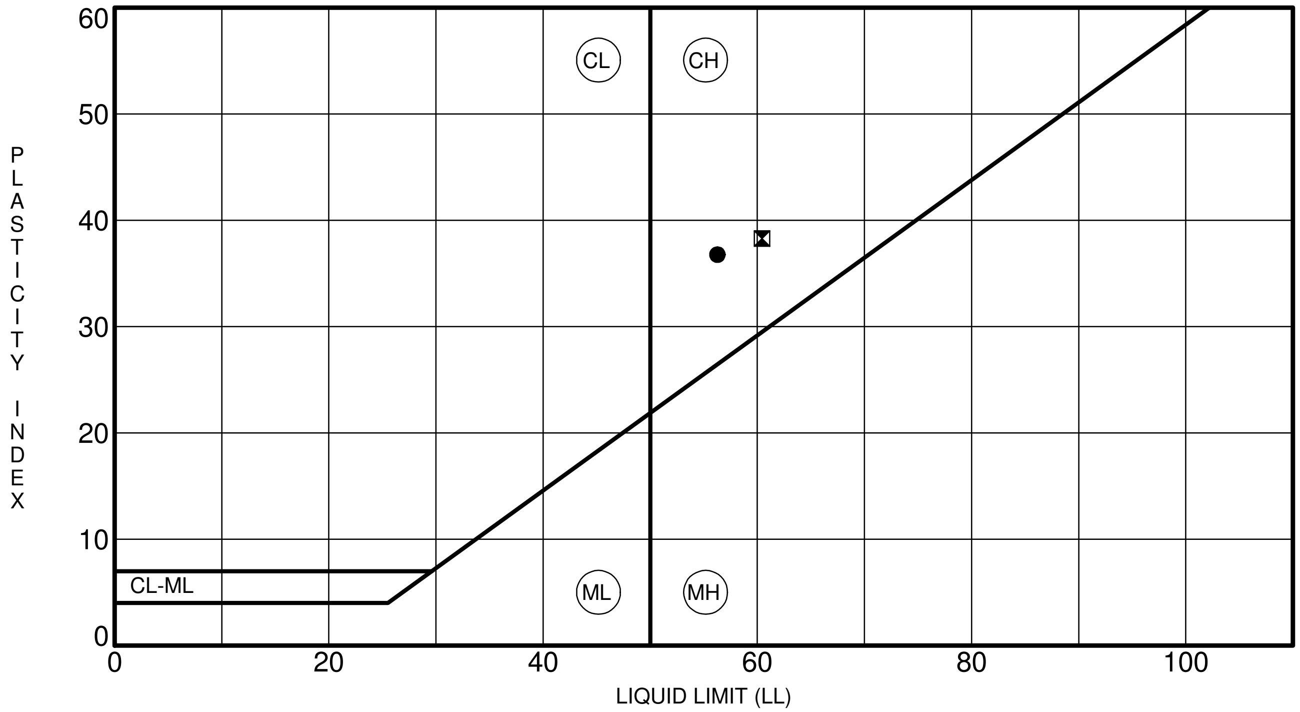
FILE NO. **PG5161**

DATE **15 Apr 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 5-21**	**SS 5**	**56**	**19**	**37**		**CH - Inorganic clays of high plasticity**
☒	**BH 6-21**	**SS 5**	**60**	**22**	**38**		**CH - Inorganic clays of high plasticity**

CLIENT **Exit 96 Developments**

FILE NO. **PG5161**

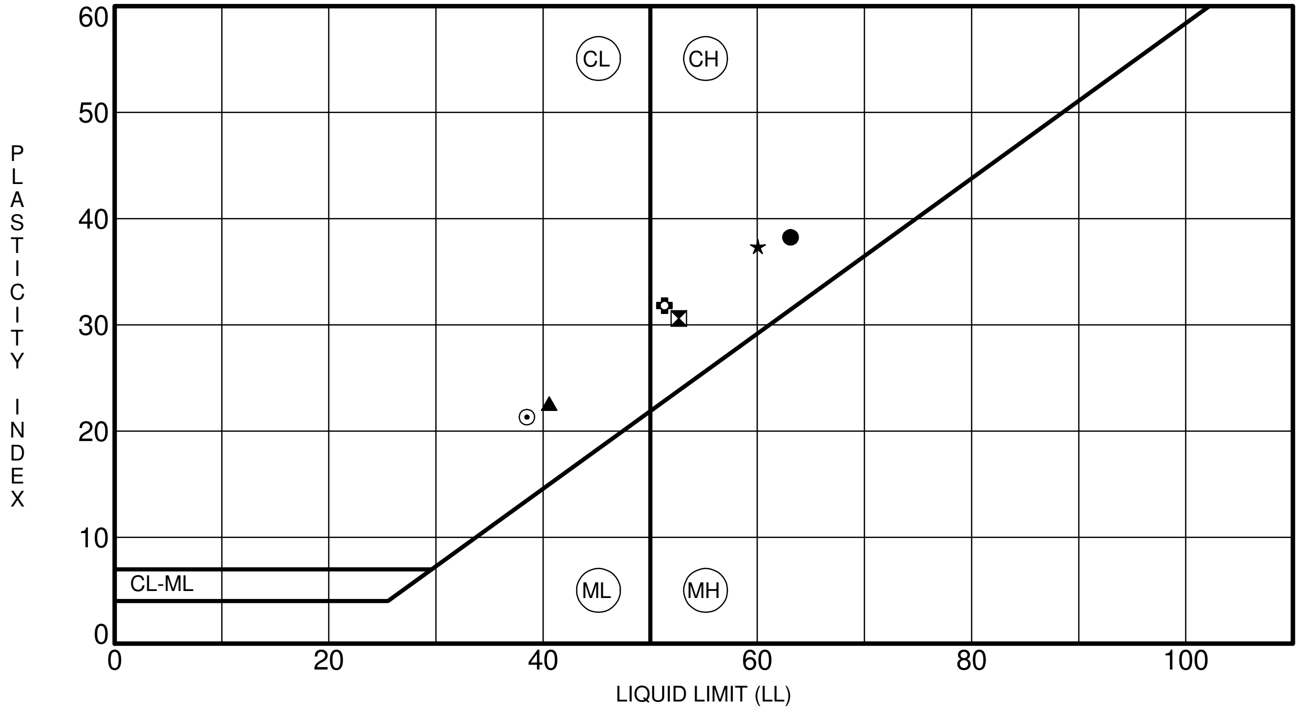
PROJECT **Supplemental Geotechnical Investigation - Proposed Warehouse Development - Thunder Road**

DATE **14 Jul 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS



Specimen Identification			LL	PL	PI	Fines	Classification
●	**BH 1-20**	**SS 2**	**63**	**25**	**38**		**CH - Inorganic clays of high plasticity**
⧗	**BH 2-20**	**SS 2**	**53**	**22**	**31**		**CH - Inorganic clays of high plasticity**
▲	**BH 3-20**	**SS 2**	**41**	**18**	**23**		**CL - Inorganic clays of low plasticity**
★	**BH 4-20**	**SS 2**	**60**	**23**	**37**		**CH - Inorganic clays of high plasticity**
⊙	**BH 5-20**	**SS 2**	**38**	**17**	**21**		**CL - Inorganic clays of low plasticity**
✿	**BH 6-20**	**SS 3**	**51**	**20**	**32**		**CH - Inorganic clays of high plasticity**

CLIENT **Exit 96 Developments**

FILE NO. **PG5161**

PROJECT **Geotechnical Investigation - Prop. Warehouse Development - Thunder Road**

DATE **30 Jun 20**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

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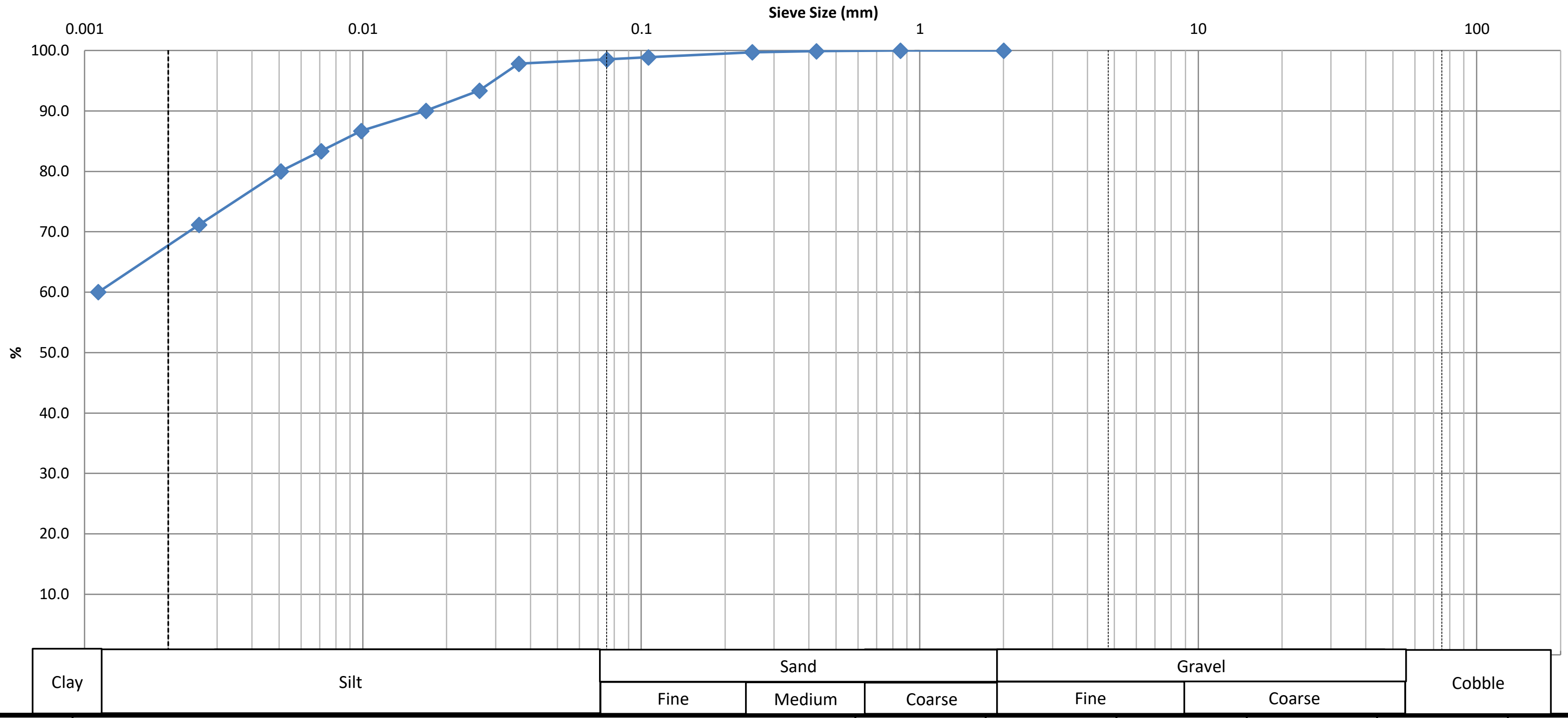
SIEVE ANALYSIS ASTM C136

CLIENT:	Exit 96 Developments	DEPTH:	5' - 7'	FILE NO:	PG5161
CONTRACT NO.:		BH OR TP No.:	BH6 SS3	LAB NO:	18125
PROJECT:	Thunder Road @ Boundary Road			DATE RECEIVED:	22-Jul-20
				DATE TESTED:	23-Jul-20
DATE SAMPLED:	22-Jul-20			DATE REPORTED:	0-Jan-00
SAMPLED BY:	A.C.			TESTED BY:	DB



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					40.3					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					0.0	1.5	28.0	70.5		

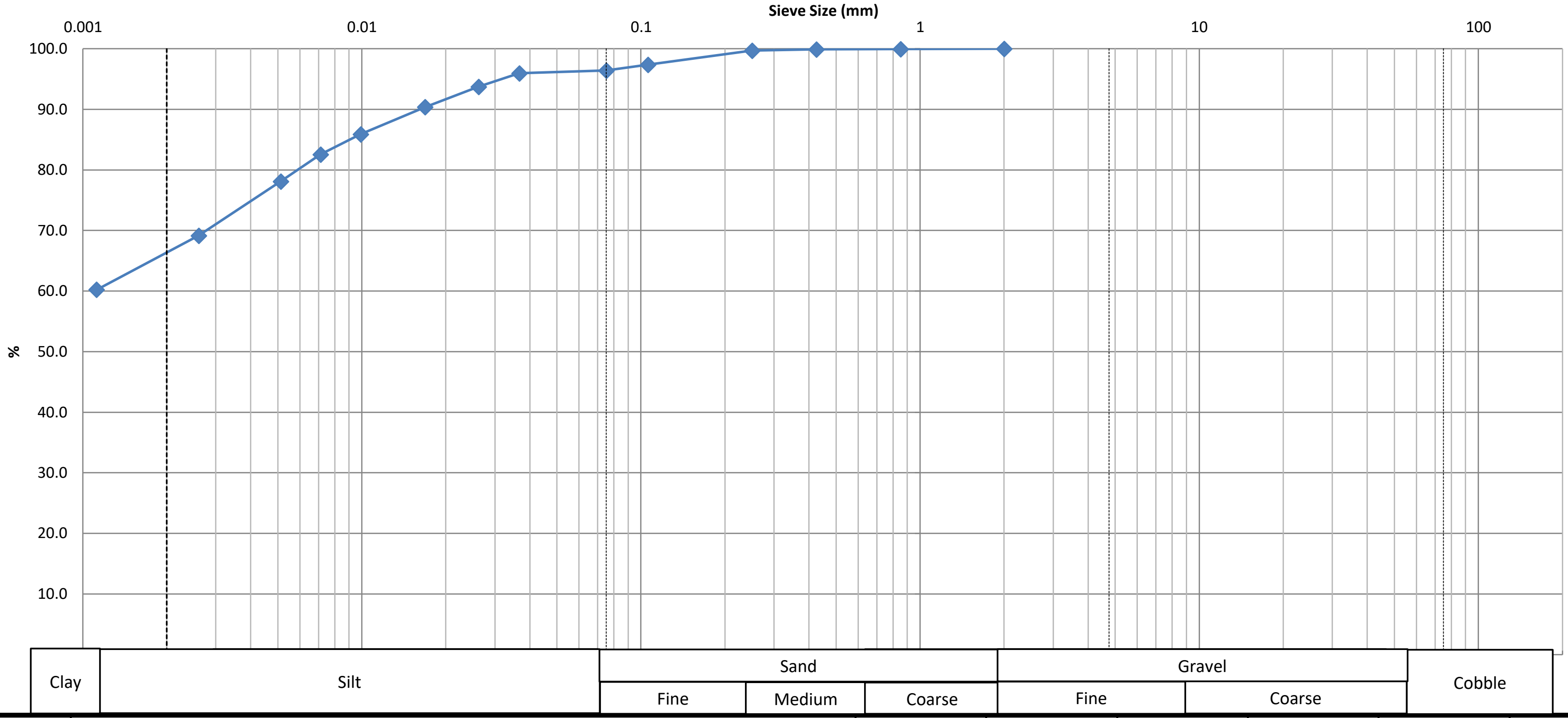
Comments:

REVIEWED BY:	Curtis Beadow	Joe Fosyth, P. Eng.

patersongroup
consulting engineers

SIEVE ANALYSIS ASTM C136

CLIENT:	Exit 96 Developments	DEPTH:	5' - 7 '	FILE NO:	PG5161
CONTRACT NO.:		BH OR TP No.:	BH2 SS3	LAB NO:	18126
PROJECT:	Thunder Road @ Boundary			DATE RECEIVED:	22-Jul-20
				DATE TESTED:	23-Jul-20
DATE SAMPLED:	22-Jul-20			DATE REPORTED:	1-Aug-20
SAMPLED BY:	A.C.			TESTED BY:	DB



Identification	**Soil Classification**					**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
						38.7					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					0.0	3.6		25.9		70.5	

Comments:

REVIEWED BY:	**Curtis Beadow**	**Joe Fosyth, P. Eng.**

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 30331

Report Date: 14-Jul-2020

Order Date: 8-Jul-2020

Project Description: PG5161

	Client ID:	BH6-SS3	-	-	-
	Sample Date:	02-Jul-20 11:00	-	-	-
	Sample ID:	2028331-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	65.5	-	-	-
General Inorganics					
pH	0.05 pH Units	8.28	-	-	-
Resistivity	0.10 Ohm.m	30.3	-	-	-
Anions					
Chloride	5 ug/g dry	17	-	-	-
Sulphate	5 ug/g dry	58	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5161-1 - TEST HOLE LOCATION PLAN

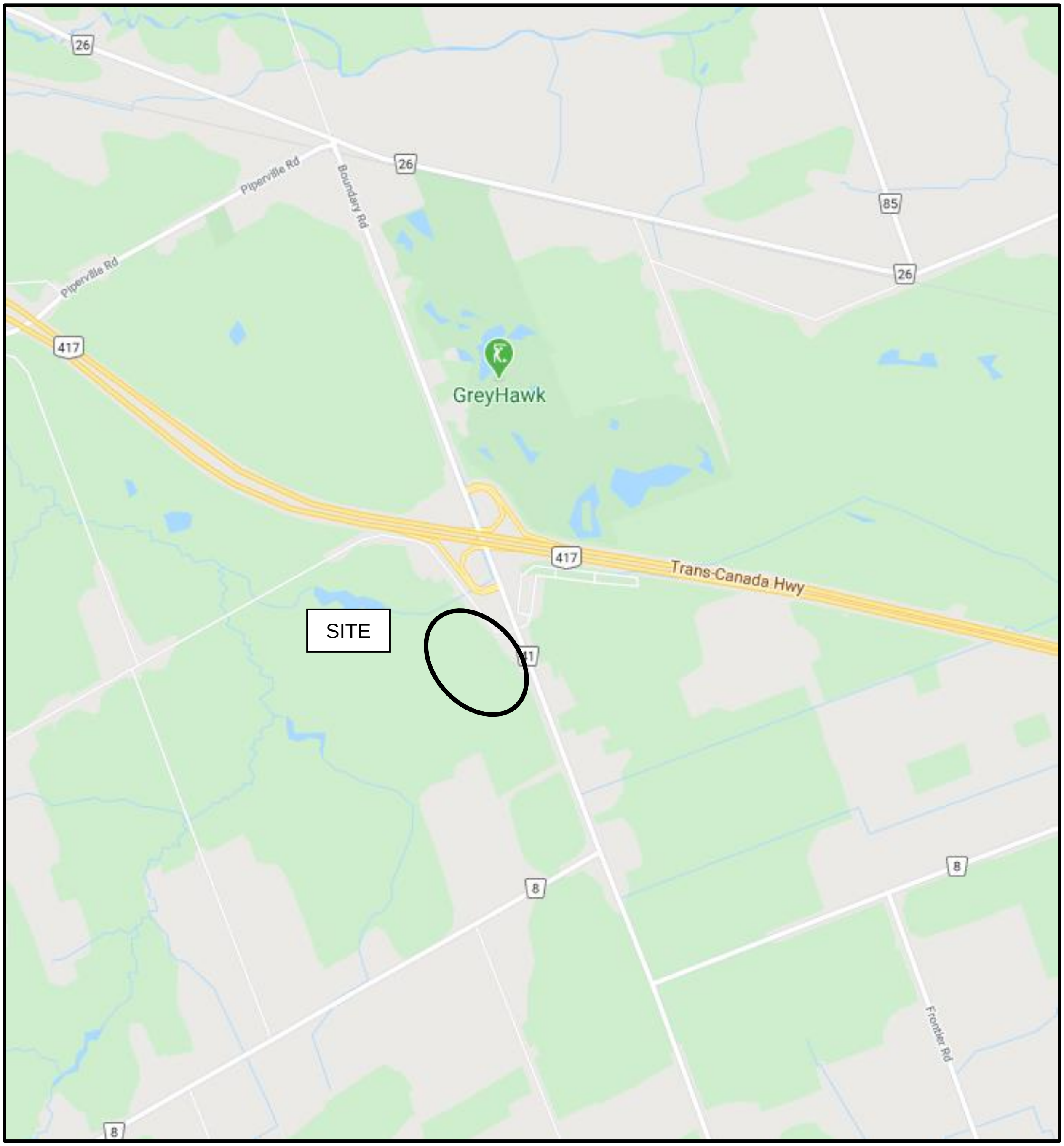
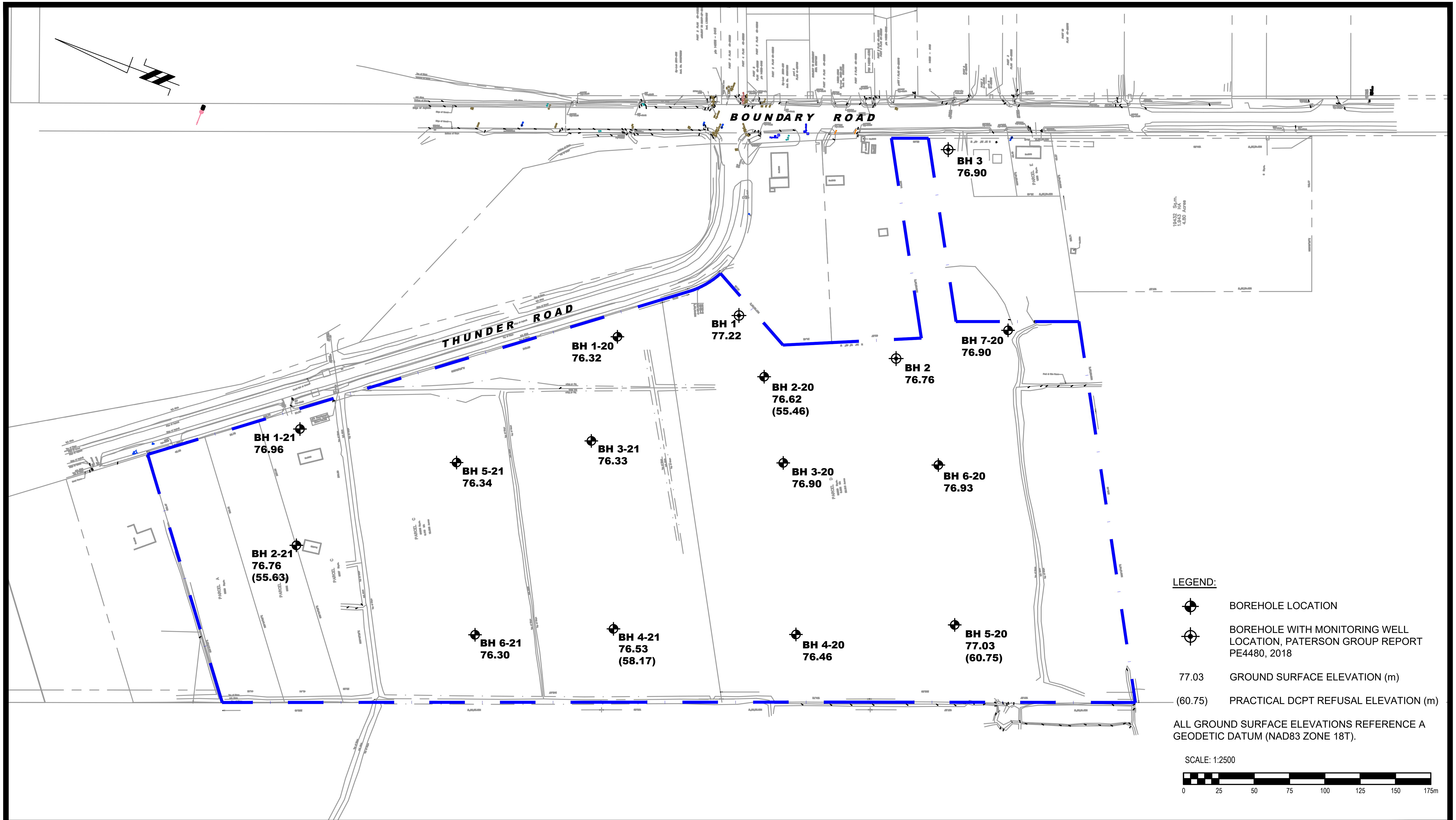


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⊕	BOREHOLE LOCATION
⊕	BOREHOLE WITH MONITORING WELL LOCATION, PATERSON GROUP REPORT PE4480, 2018
77.03	GROUND SURFACE ELEVATION (m)
(60.75)	PRACTICAL DCPT REFUSAL ELEVATION (m)

ALL GROUND SURFACE ELEVATIONS REFERENCE A GEODETIC DATUM (NAD83 ZONE 18T).

patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
2	BH 3-21 AND BH 6-21 ADDED	19/07/2021	JV
1	BH 1-21 AND BH 2-21 ADDED	21/04/2021	JV

AVENUE 31
GEOTECHNICAL INVESTIGATION
6150 THUNDER ROAD AND 5368 BOUNDARY ROAD
OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:2500
Date: 08/2020
Drawn by: NFRV
Report No.: PG5161-1
Checked by: JV
Approved by: DJG

PG5161-1

Revision No.: 2

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