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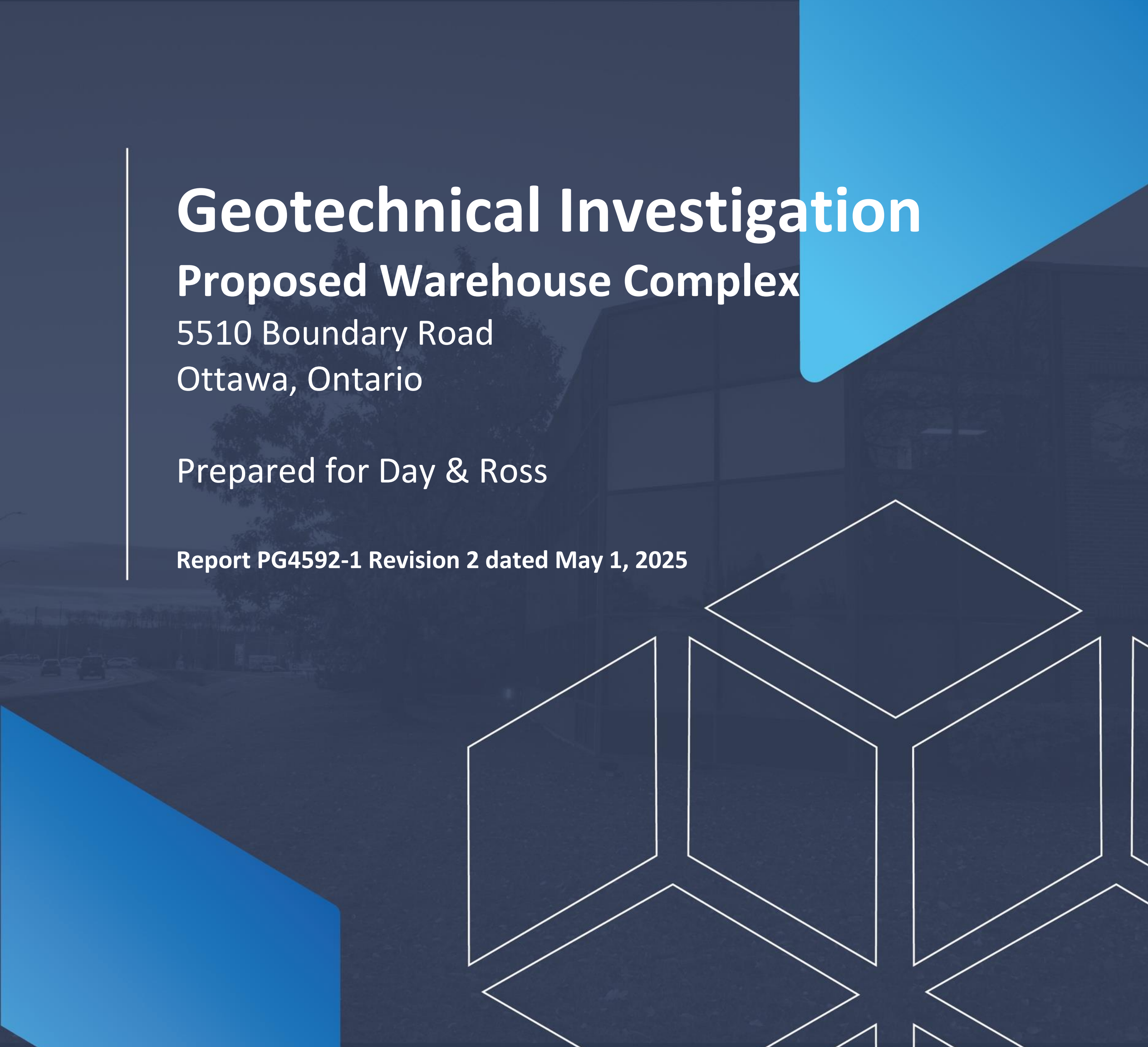

Geotechnical Investigation

Proposed Warehouse Complex

5510 Boundary Road
Ottawa, Ontario

Prepared for Day & Ross

Report PG4592-1 Revision 2 dated May 1, 2025

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Day & Ross to conduct a geotechnical investigation for the proposed warehouse complex to be located at 5510 Boundary Road in the City of Ottawa (reference should be made to Figure 1 – Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

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

☐ 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.

Investigating the presence or potential presence of contamination on the subject site was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a slab-on-grade, single-storey warehouse building with limited office space. It is further understood that a firefighting water storage tank(s) will be placed on a below-grade slab below the office portion of the warehouse building. The warehouse will be surrounded by associated access lanes, loading docks, and parking areas. 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 Program

The field program for the geotechnical investigation was carried out between August 1 and August 3, 2018, and consisted of a total of four (4) boreholes (BH 1 through BH 4) advanced to a maximum depth of 9.75 m below the existing ground surface and twelve (12) test pits (TP 1 through TP 12) advanced to a maximum depth of 3.5 m below existing grade.

The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access. The locations of the test holes are shown on Drawing PG4592-1 – Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted drill rig operated by a two-person crew and the test pit procedure consisted of excavating to the required depths at the selected locations and sampling the overburden using a hydraulic shovel. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The test hole procedure consisted of augering or excavating to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

The soil samples were collected from the boreholes using a 50 mm diameter split-spoon (SS) sampler, a Shelby tube (TW), or from the drill auger flights. Test pit samples were collected at selected intervals from the test pit sidewalls. The samples were initially classified on site, placed in sealed plastic bags, and transported to our laboratory. The depths at which the drill auger, split-spoon, Shelby tubes, and grab samples were recovered from the boreholes and test pits are shown as AU, SS, TW, and G, 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 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 was carried out in cohesive soils using a field vane apparatus.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) completed at BH1 during the field program. 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. Due to the low resistance exerted by the silty clay in some boreholes, the cone was often pushed using the hydraulic head of the drill rig until resistance to penetration was encountered. The hammer was then used to further advance the cone to practical refusal.

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

Groundwater

A 32 mm diameter monitoring well was installed in BH 4-18 and flexible polyethylene standpipes were installed in BH 1-18, BH 2-18, and BH 3-18, to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. Open hole groundwater infiltration levels were observed at the time of excavation at each test pit location. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities.

The test hole locations, and the ground surface elevation at each test hole location, were surveyed by Annis, O'Sullivan, Vollebekk Ltd. The locations of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG4592-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

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 three (3) Shelby tube samples were submitted for unidimensional consolidation testing from the boreholes completed during our investigation. The results of the consolidation testing are presented on the Unidimensional Consolidation Test Results sheets presented in Appendix 1 and are further discussed in Subsection 5.4.

4.0 Observations

4.1 Surface Conditions

The subject site is currently undeveloped and generally covered with grass, small brush, forested areas, and scattered fill piles. Two gravel covered access points were noted along the east portion of the site. The ground surface across the subject site is relatively flat and at grade with the adjacent Boundary Road.

Water ponds were noted along the eastern central portion and norther borderline of the site. A small drainage ditch was noted along the center of the site running south-north and a drainage swale noted bordering the site along the east border line. Based on our review of the historical aerial maps, the center of the site was previously a water ponded area which was in-filled with miscellaneous fill material.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the subject site consists of topsoil and/or fill underlain by silty sand and further by a deposit of brown to grey silty clay.

The topsoil layers thickness was observed to range between 50 to 80 mm. Fill, consisting of brown silty clay with variable amounts of sand, crushed stone, organics, and cobbles, was encountered at BH 1 through BH 4, and TP 1 through TP 12. The fill layer was observed to start at either ground surface, or below the topsoil layer, and would extend to depths ranging from 1.8 to 3.5 m below ground surface.

A very loose to compact, brown silty sand deposit was encountered at BH 1 through BH 3, TP 1 through TP 7, and TP 10 through TP 12. The sand layer was observed at depths ranging between 1.4 to 3.5 m below ground surface. The above-noted layers were underlain by a deposit of soft to firm, brown to grey silty clay. The grey silty clay layer was observed to begin at depths ranging between 2.4 to 3.3 m and is inferred to extend down to the bedrock surface due to the low resistance observed during the DCPT.

Practical refusal to DCPT was encountered at BH 1 at a depth of 24.0 m below the existing ground surface.

Reference should be made to the Soil Profile and Test Data Sheets in Appendix 1 for details of the soil profile encountered at each borehole location.

Bedrock

Based on available geological mapping, bedrock in the area of the subject site consists of shale of the Carlsbad Formation with a drift thickness ranging between 25 to 35 m.

4.3 Groundwater

Groundwater levels were measured in the monitoring well and piezometers at the borehole locations on August 8, 2018. The measured groundwater levels are presented on the Soil Profile and Test Data sheets in Appendix 1, and in Table 1 below.

Table 1 – Measured Groundwater Levels

Test Hole Number	Method	Ground Surface Elevation (m)	Measured Groundwater Level		Date
			Depth (m)	Elevation (m)	
BH 1	Monitoring Well	78.29	1.30	77.00	August 8, 2018
BH 2	Piezometer	78.67	6.49	72.18	August 8, 2018
BH 3	Piezometer	77.67	1.29	76.38	August 8, 2018
BH 4	Piezometer	77.98	1.08	76.90	August 8, 2018
Summary of Open Hole Groundwater Level (TP)					
TP 1	Observed	77.87	3.10	74.77	August 1, 2018
TP 2	Observed	77.56	0.60	76.96	August 1, 2018
TP 3	Observed	77.51	3.30	74.21	August 1, 2018
TP 4	Observed	77.66	0.75	76.91	August 1, 2018
TP 5	Observed	77.93	2.20	75.73	August 1, 2018
TP 6	Observed	78.45	3.00	75.45	August 1, 2018
TP 7	Observed	78.00	2.10	75.90	August 1, 2018
TP 8	Observed	78.00	Dry	n/a	August 1, 2018
TP 9	Observed	78.17	0.65	77.52	August 1, 2018
TP 10	Observed	78.27	Dry	n/a	August 1, 2018
TP 11	Observed	78.07	3.10	74.97	August 1, 2018
TP 12	Observed	76.25	2.60	73.65	August 1, 2018

NOTE: The ground surface elevations at the test hole location of the current investigation were surveyed by Annis, O'Sullivan, Vollebekk, Ltd. and are assumed to have been referenced to a geodetic datum.

It should be noted that surface water can become trapped within a backfilled borehole, which can lead to higher than typical groundwater level observations. Similarly, it is our experience that surface water generated by snowmelt and rainfall events may sheet drain into the borehole column given the relatively impermeable nature of the silty clay soil surface.

The long-term groundwater level can also be estimated based on the observed colour, moisture content, and consistency of the recovered samples. Based on these observations, the long-term groundwater level is expected at a depth of 2 to 3 m below the existing grade, corresponding to an approximate geodetic elevation of **75 to 76 m**.

It should be noted that groundwater levels are subject to seasonal fluctuations, therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Options

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed warehouse building, including the below-grade slab for the firefighting water storage tank(s), can be founded on conventional shallow foundation placed on a reinforced granular pad, bearing on an undisturbed, native soil, or an existing fill (proof-compacted and approved by Paterson) bearing medium, or on end bearing piles.

Any light duty external structures may be founded on conventional spread footings placed on an undisturbed, compact silty sand, an undisturbed firm silty clay deposit, or engineered fill placed over an undisturbed, native bearing medium.

Permissible Grade Raise

Due to the presence of a silty clay deposit, the site will be subject to a permissible grade raise restriction. It should be noted that a fill layer ranging in depth between 1.8 to 3.1 m below existing grade was noted during the field investigation. Based on our review of the available historical aerial mapping, for the past 20 years, the site has been used as a fill placement area. Therefore, due to the significant thickness of fill encountered across the site, the permissible grade raise restrictions were calculated based on the native soil surface.

Lightweight Fill Recommendations

Where the proposed grading exceeds the permissible grade raise restriction, and where a surcharge program has not been completed, lightweight fill (LWF) will be required for settlement-sensitive structures, including a combination of granular materials and LWF (EPS Type 19 blocks) below proposed floor slabs, to manage the long-term settlements due to floor slabs behaving independently from main building structures.

Where a surcharge program has been completed, the requirement for lightweight fill will be reduced.

Lightweight fill recommendations can be provided once the surcharge program has been completed and finalized grading plan is available.

Existing Fill - Original Versus Existing Ground Surface

Based on the current findings, the original ground surface is expected to be at an approximate geodetic elevation of 76 m within the north portion of the site which slopes down to 74.5 m along the south portion of the site. It is suspected that the lower original ground surface may be associated with a drainage area since the surrounding area is relatively flat.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures.

Consideration can be given to leaving the existing fill, free of significant amounts of deleterious materials, below the proposed building floor slab outside of the lateral support zone of the proposed footings and within the proposed parking areas and access lanes. However, it is recommended that the existing fill be approved by the geotechnical consultant once the subgrade level is determined at design stage and exposed during the construction phase.

The approved existing fill material should be proof-rolled using suitable compaction equipment under dry conditions and above freezing temperatures, and reviewed by Paterson personnel. Poor performing areas should be removed and replaced with engineered fill.

Any topsoil remaining on site and fill, containing deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding, and other settlement-sensitive structures.

Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Vibration Considerations

Construction operations could cause vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels as much as possible should be incorporated into the construction operations to maintain a cooperative environment with the residents.

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of a shoring system with soldier piles or sheet piling will require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). These guidelines are for current construction standards.

Considering there are several sensitive buildings in close proximity to the subject site, consideration to lowering these guidelines is recommended. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill placed for grading beneath the building footprints should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site.

The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the buildings should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking areas where settlement of the ground surface is of minor concern. These materials 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 or below other settlement sensitive structures, it should be compacted in thin lifts to at least 95% of the material's SPMDD using a suitably sized vibratory sheepsfoot roller and under the full-time supervision of Paterson field personnel.

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.

Compacted Granular Fill Working Platform (Piled Foundation)

If consideration is given to supporting the proposed building on a pile foundation, the use of heavy equipment would be required to install the piles (i.e., pile driving crane). It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance to the underlying soil.

It is recommended that a minimum 600 mm thick layer of OPSS Granular B Type II crushed stone be placed as working platform throughout the building footprint which will be supported by piles. The working pad granular should be compacted to a minimum of 98% of its standard Proctor maximum dry density (SPMDD) in maximum 300 mm thick lifts. The existing stone fill surface can be used as the subgrade for this purpose provided it is re-compacted in place using a smooth-drum roller.

Once the piles have been driven and cut off, the working platform can be re-graded, and soil tracked in, or soil pumping up from the pile installation locations, can be bladed off and the surface can be topped up, if necessary, and re-compacted to act as the substrate for further fill placement for the slab structure.

5.3 Foundation Design

Conventional Spread Footings – Warehouse Building

Should the proposed warehouse building, including the below-grade slab for the firefighting water storage tank(s), be founded using a shallow foundation, the following foundation design will be applicable. The conventional spread footings, designed using the bearing resistance values presented below, should be placed over a 600 mm thick Granular A or B Type II engineered fill pad, wrapped in a biaxial geogrid. The reinforced granular pad should extend a minimum of 1.5 m beyond the interior and exterior footing faces. A geotextile should be placed between the reinforced granular pad and the bearing subgrade surface (approved fill material or undisturbed, native soil).

The reinforced pads should be placed over a native, undisturbed, firm, brown to grey silty clay, or an approved, proof-compacted existing fill surface, reviewed and approved by Paterson.

Footings placed on an engineered fill pad, constructed in accordance with the recommendations above, over an approved, proof-compacted existing fill surface, or native, firm silty clay, can be designed using a bearing resistance value at an SLS of **100 kPa** and a factored bearing resistance value at a ULS of **150 kPa**.

A geotechnical resistance factor of 0.5 is applied to the above noted bearing resistance values at ULS. The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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, in the dry, prior to the placement of concrete for footings.

Conventional Spread Footings – Light-Duty External Structures

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, firm, brown to grey silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **60 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **100 kPa**.

Conventional shallow spread footing foundations placed on an undisturbed, compact, brown silty sand bearing surface can be designed using a bearing resistance value at an SLS of **80 kPa** and a factored bearing resistance value at a ULS of **140 kPa**.

Footings placed on compacted engineered fill (proof-compacted and approved by Paterson), placed on a native, undisturbed soil bearing surface, can be designed using a bearing resistance value at an SLS of **100 kPa** and a factored bearing resistance value at a ULS of **150 kPa**.

A geotechnical resistance factor of 0.5 is applied to the above noted bearing resistance values at ULS.
The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

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, in the dry, prior to the placement of concrete for footings.

Proof Rolling and Subgrade Improvement for Loose Sand Below Footings

Where the sand bearing surface for footings is considered loose by Paterson at the time of construction, it would be recommended to proof roll (i.e., re-comapct) the bearing surface prior to forming for footings. Proof-rolling is recommended to be undertaken in **dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimal compaction levels.

The proof rolling compaction program should be reviewed and approved by Paterson at the time of construction. Depending on the looseness and degree of saturation of loose sandy soils at the time of construction, other measures (additional compaction, sub-excavation and reinstatement of crushed stone fill, mud slab) may be recommended to accommodate site conditions at the time of construction. However, these considerations would be evaluated at the time of design by Paterson on a footing-specific basis.

Deep Foundations (End Bearing Piles)

A deep foundation method, such as end bearing piles, may also be considered for the foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored 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. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended.

This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values.

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving.

A full-time field review program should be conducted by Paterson field personnel during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical).

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	1,495	25	40
245	11	1,750	24	48.5
245	131	2,000	25	56

The minimum recommended centre-to-centre pile spacing is 3 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. It is recommended that Paterson undertake the associated pile testing efforts in conjunction with the piling subcontractor at the construction stage.

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 approximately 21 to 26 m of 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. As such, the estimated downdrag load for each 245 mm diameter pile is anticipated to vary from 325 kN for a 21 m pile to 400 kN for a 26 m pile.

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 19, which has a compressive strength at 5% deformation of 90 kPa.

Should additional passive resistance be required, 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 I really counteracting a dead load, in our opinion, 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 tanδ	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
Granular Working Surface (Coarse Open-Graded Crushed Stone)	19.0	12.0	36	0.55	0.26	0.41	7.5
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 earth coefficients incorporate wall friction of 0.5 φ'

Settlement

The total and differential settlements associated with the footing loading conditions using the bearing resistance value at SLS provided are estimated to be 25 and 20 mm, respectively, for structures designed using the above-noted foundation design.

Lateral Support

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 a silty clay bearing medium 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 or engineered fill of the same or higher capacity as the bearing soil.

5.4 Settlement Surcharge Monitoring Program

To reduce and/or remove the requirements for lightweight fill (LWF) within portions of the proposed development (where the proposed grading exceeds the permissible grade raise restriction), including the proposed warehouse complex, a settlement surcharge monitoring program was implemented for the subject site where significant permissible grade raise exceedances occur.

The surcharge program was initiated in December 2024, following the placement of the bulk of the fill material for the surcharge pile. At that time, four (4) settlement plates (SP 1-24, SP 2-24, SP 3-24, and SP 4-24) were installed to permit ongoing monitoring of the surcharge program. A temporary benchmark (TBM) was installed and assigned a geodetic elevation by Paterson personnel, using a high-precision GPS unit, to provide a geodetic elevation reference for the settlement plate elevations.

5.5 Permissible Grade Raise Recommendations

Consideration must be given to potential settlements which could occur due to the presence of the silty clay deposit and the combined loads from the proposed footings, any groundwater lowering effects, and grade raise fill. The foundation loads to be considered for the settlement case are the continuously applied loads which consist of the unfactored dead loads and the portion of the unfactored live load that is considered to be continuously applied.

Consolidation Testing

Generally, the potential long-term settlement is evaluated based on the compressibility characteristics of the silty clay. These characteristics are estimated in the laboratory by conducting unidimensional consolidation tests on undisturbed soil samples collected using Shelby tubes in conjunction with a piston sampler. A total of 3 site specific consolidation tests were completed for this project. The results of the consolidation tests are included in Appendix 1.

Value p'_c is the preconsolidation pressure of the sample and p'_o is the effective overburden pressure. The difference between these values is the available preconsolidation. The increase in stress on the soil due to the cumulative effects of the fill surcharge, the footing pressures, the slab loadings and the lowering of the groundwater should not exceed the available preconsolidation if unacceptable settlements are to be avoided.

The values C_{cr} and C_c are the recompression and compression indices, respectively, and are a measure of the compressibility of the soil due to stress increases below and above the preconsolidation pressures. The higher values for the Cc, as compared to the C_{cr}, illustrate the increased settlement potential above, as compared to below, the preconsolidation pressure.

It should be noted that the values of p'_c, p'_o, C_{cr} and C_c are determined using standard engineering practices and are estimates only. In addition, natural variations within the soil deposit would also affect the results. Furthermore, the p'_o parameter is directly influenced by the groundwater level. While the groundwater levels were measured at the time of the fieldwork, the levels vary with time and this has an impact on the available preconsolidation. Lowering the groundwater level increases the p'_o and therefore reduces the available preconsolidation.

Unacceptable settlements could be induced by a significant lowering of the groundwater level. The p'_o values for the consolidation tests carried out for the present investigation are based on the long-term groundwater level being 0.5 m above the bottom of the silty clay crust. The level of the groundwater level is based on the colour and undrained shear strength profile of the silty clay.

Table 4 – Summary of Consolidation Test Results

Borehole No.	Sample	Sample Depth (m)	p'_c (kPa)	p'_o (kPa)	C_{cr}	C_c	Q (*)
BH 2	TW 6	5.05	64.7	27.8	0.047	1.293	A
BH 3	TW 5	3.38	49.5	16.3	0.048	1.986	A
BH 4	TW 5	5.69	82.8	31.8	0.058	3.722	G

* - Q - Quality assessment of sample - G: Good A: Acceptable P: Likely disturbed

Settlement

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

Permissible Grade Raise

Preliminary permissible grade raise recommendations were 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 **1.0 to 1.2 m** is recommended for slab-on-grade at the site, using 400 mm of EPS geofoam blocks (Type 19 or higher), to provide compensation for the sustained slab-on-grade loading.

For parking and loading areas away from the building foundation, preliminary permissible grade raise recommendations will be slightly higher at **1.4 m**.

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).

5.6 Design for Earthquakes

Seismic shear wave velocity testing was completed for the subject site to accurately determine the applicable seismic site classification for the proposed building in accordance with Table 4.1.8.4.A of the Ontario Building Code 2024. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figure 2 and Figure 3 presented in Appendix 2 of this report.

Field Program

The seismic array testing location was placed as presented in Drawing PG4592-1 – Test Hole Location Plan, attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 3 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio.

The shot locations were also completed in a forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 20, 4.5 and 3 m away from the first and last geophone, and at the center of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results was completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the foundation of the building. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Based on our testing results, the average overburden shear wave velocity is **168.7 m/s**, while the bedrock shear wave velocity is **1,500 m/s**. The overburden thickness below underside of footing is assumed to be 22.4 m, based on an underside of footing at 1.5 m below ground surface.

The V_{s30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2024 and as presented below:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{23.5\ m}{168.7\ m/s} + \frac{6.5\ m}{1{,}500\ m/s}\right)}$$

$$V_{s30=}\ 209\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building placed on overburden material is **209 m/s**. Therefore, a **Site Class D** is applicable for the design of buildings placed on overburden materials and as per Table 4.1.8.4.A of the OBC 2024. The soils underlying the subject site are not susceptible to liquefaction.

5.7 Slab on Grade Construction

Under Slab Materials

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 prepare the profile, provided below, for slab-on-grade construction. Any soft areas should be removed and backfilled with appropriate backfill material.

It is recommended that the sub-slab profile consists of the following:

☐ A thickness yet to be determined of EPS Blocks Type 19 (LWF) overlying the native soil subgrade.

☐ Geotextile or poly separation layer overlying the EPS Blocks.

☐ 400 to 450 mm of OPSS Granular A placed over the separation layer for the support of the slab-on-grade.

Modulus of Subgrade Reaction

Typical values of subgrade modulus for the OPSS Granular A, native silty sand and native silty clay surfaces are provided in Table 5.

Table 5 – Modulus of Subgrade Reaction

Soil Type	Modulus of Soil Reaction (MPa/m)
OPSS Granular A Subgrade	20
Silty Sand	15
Silty Clay Deposit	5

5.8 Pavement Design

Minimum Pavement Structure Recommendations

For the pavement structures, it’s expected that concrete aprons will be used along the loading docks along with dolly pads. The loading docks will most likely be 1.2 m lower than the floor slab. The remainder of the pavement structures will be flexible asphaltic concrete and will be able to comply with the permissible grade raise restrictions provided. The proposed pavement structures are presented in Tables 6 and 7.

Table 6 – Recommended Pavement Structure – Parking Stalls and Light-Duty Traffic Area

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 fill or in-situ soil.	

Table 7 – Recommended Pavement Structure – Access Lanes and Heavy Truck Parking Areas

Thickness (mm)	Material Description
40	**Wear Course** – Superpave 12.5 Asphaltic Concrete
50	**Binder Course** – 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 placedover fill or in situ soil.	

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 I or II material. Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment.

Crushed Concrete for Subbase Layer

It is understood that crushed concrete is introduced as a replacement for the recommended OPSS Granular B Type II subbase layer. From a geotechnical perspective, the use of crushed concrete is acceptable. However, the pavement structure design must be altered to optimise the performance of the new subbase, as follows:

❑ The subbase layer must be increased by 100 mm and should consist of free draining and compacted to a minimum of 98% of the material’s SPMDD.

❑ The crushed concrete used should be a minimum of 50 mm in size, well graded and free of any deleterious materials such as, rebar debris and metal.

It should be noted that although the usage of crushed concrete is acceptable for the subbase layer, the crushed concrete will not mirror all the characteristics of the recommended OPSS Granular B Type II. Due to the nature of the concrete, the concrete partially loses its strength when exposed to excessive water over the long run. Also, due to the shallow groundwater across the subject site, it is recommended that the subgrade have a positive drainage to direct any water beneath the pavement to subdrains that will direct the water to positive outlets. The pavement structure drainage recommendations are discussed in the following section.

Periodic granular size distribution testing must be completed during the construction of the parking area by Paterson personnel to confirm the sufficiency of the crushed concrete used within the subbase layer.

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 extend in four orthogonal directions and longitudinally when placed along a curb. The clear crushed stone surrounding the drainage lines or the pipe, 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

Since hardscaping is anticipated around the perimeter of the proposed slab-on-grade structure, it is recommended to implement a perimeter foundation drainage system around the entire building perimeter. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock and surrounded on all sides by 150 mm of 10 mm clear crushed stone. The pipe should be placed at the footing level around the exterior perimeter of the structure. The clear stone should be wrapped in a non-woven geotextile. 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 drainage geocomposite, such as Delta Drain 6000, 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.

Concrete Sidewalks Adjacent to Buildings

It is recommended that the upper 600 mm of backfill placed below the concrete sidewalks adjacent to the building footprints to consist of non-frost susceptible material such as OPSS Granular A or Granular B Type II. The sidewalks should be underlain by a layer of rigid insulation at entranceways to minimize the potential for the sidewalks to raise in response to frost migration within the subgrade soils.

The granular material should be placed in maximum 300 mm loose lifts and compacted to a minimum of 98% of the material’s SPMDD using suitable compaction equipment. The subgrade material should be shaped to promote positive drainage towards the building’s perimeter drainage system. Consideration should be given to placing a layer of rigid insulation below the granular fill layer, however, should be detailed by Paterson once design drawings are being complete by others.

6.2 Protection Against Frost Action

Foundation Structures

Perimeter footings and/or pile caps and grade beams of heated structures are required to be insulated against the deleterious effects of frost action. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided for adequate frost protection of heated structures.

Exterior unheated footings, such as those for isolated exterior piers, loading docks, or footings near garage bay doors that will be exposed to exterior conditions for extended periods of time, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertake by open-cut methods (i.e., unsupported excavations).

Unsupported Excavations

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsoil at this site is considered to be mainly Type 2 and Type 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

It is recommended that a trench box be used at all times to protect personnel working in trenches. Based on this, trench boxes should be considered for all sewer pipe installations undertaken throughout the subject site.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

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

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.

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 99% of its SPMDD. The bedding material should extend at least to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 99% of its SPMDD.

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. Wet site-generated fill, such as the grey silty clay, will be difficult to re-use, as the high-water contents make compacting impractical without an extensive drying period.

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.

Clay Seals

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in service trenches that are within the silty clay desposit. The clay seals should be at least 1.5 m long in the trench direction and should extend from trench wall to trench wall. Generally, the clay seals should extend from the frost line and fully penetrate the bedding, sub-bedding and cover material.

The clay seals should consist of relatively dry and compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material’s SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches. Paterson field personnel should review the placement of all clay seals undertaken at the time of construction.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. The contractor should be prepared to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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

For typical ground or surface water volumes being pumped during the construction phase, typically 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 Persons as stipulated under O.Reg. 63/16.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site 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 using straw, propane heaters and 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, pile caps and/or grade beams are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Provisions should also be carried for accommodating spring-thaw conditions when subgrade conditions for pavements and other works are impacted by higher degrees of soil saturation.

Additional information should be provided by Paterson for planning winter construction and pavement works.

Under winter conditions, if snow and ice is present within imported fill below future building slabs, or if fill is subject to freezing conditions, then settlement of the fill should be expected, and support of a future building slab will be negatively impacted and could undergo settlement during spring and summer time conditions. Paterson should complete periodic inspections during fill placement to ensure that snow and ice quantities do not impact fill placed in settlement-sensitive areas.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

☐ Review preliminary and detailed grading, servicing and landscaping plans from a geotechnical perspective.

☐ Review of LWF recommendations and design.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

☐ Observation of all bearing surfaces prior to the placement of concrete.

☐ Observation of the proof-compacting program for the reinforced granular pad(s), if applicable.

☐ Sampling and testing of the concrete and fill materials.

☐ 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 and follow-up field density tests to determine the level of compaction achieved.

☐ 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 the completion of a satisfactory inspection program undertaken by Paterson.

All excess soil that will be transported off site must be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management*.

8.0 Statement of Limitations

The recommendations provided are in accordance with the present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests 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 the Day & Ross, or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

Owen R. Canton, B.Eng.

LICENSED PROFESSIONAL ENGINEER
May 1, 2025
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Kevin Pickard
PROVINCE OF ONTARIO

Kevin A. Pickard, P.Eng.

Report Distribution:

☐ Day & Ross
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

CONSOLIDATION TESTING RESULTS

patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

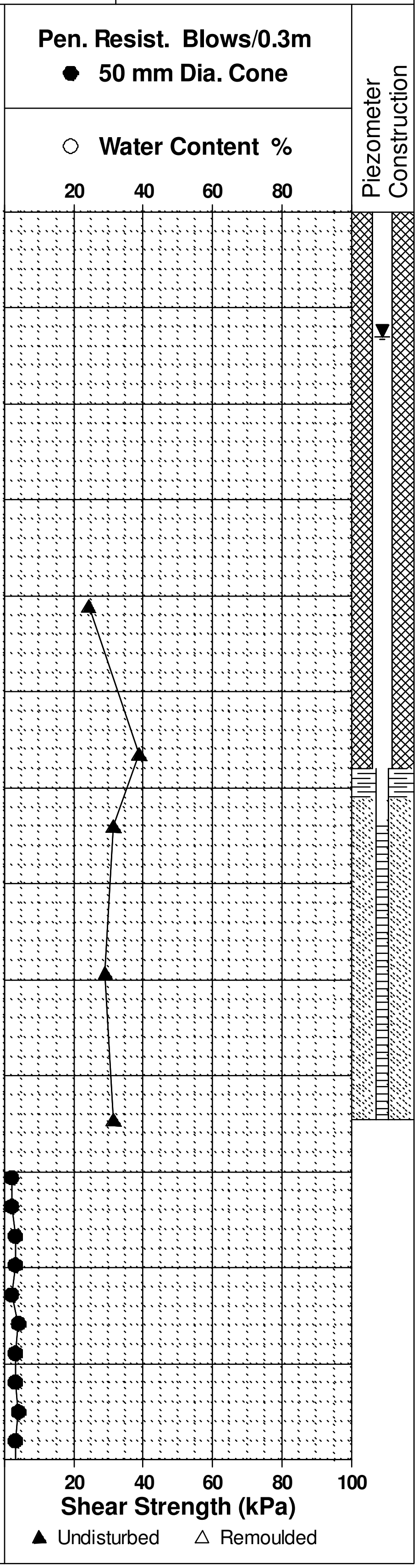
DATE 1 August 2018

FILE NO. PG4592

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay, trace sand, gravel, cobbles	▨	AU	1		
		SS	2	54	21
		SS	3	50	10
2.29					
Loose, brown **SILTY SAND**		SS	4	8	9
3.05					
Soft to firm, grey **SILTY CLAY**	▨	SS	5	88	W
		SS	6	100	6
9.45					
Dynamic Cone Penetration Test commenced at 9.45m depth.					

DEPTH (m)	ELEV. (m)
0	78.29
1	77.29
2	76.29
3	75.29
4	74.29
5	73.29
6	72.29
7	71.29
8	70.29
9	69.29
10	68.29
11	67.29
12	66.29
13	65.29



SOIL PROFILE AND TEST DATA

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REMARKS

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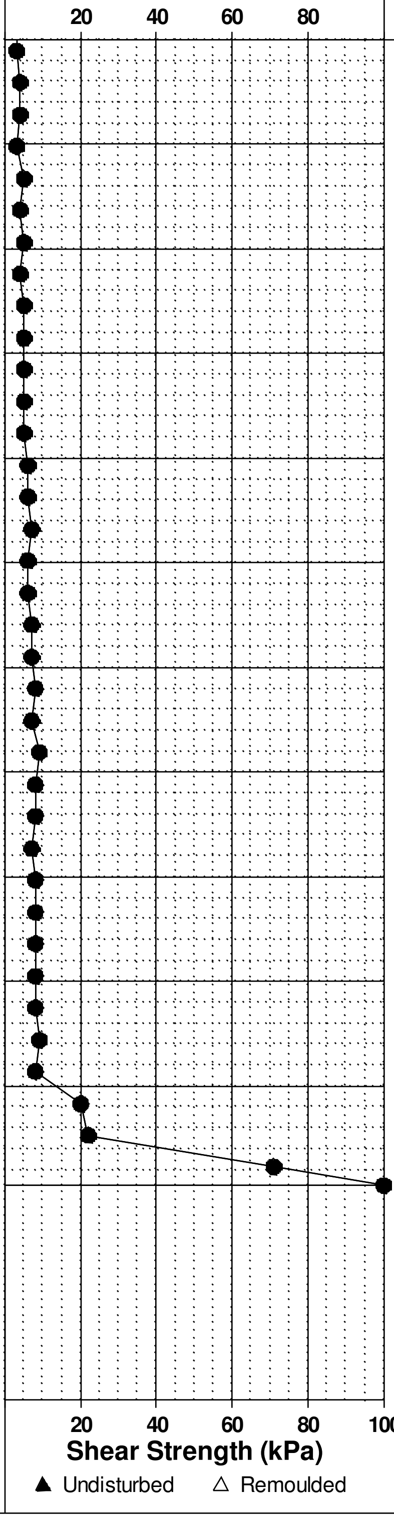
DATE 1 August 2018

FILE NO. PG4592

HOLE NO. BH 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content %	Piezometer Construction
GROUND SURFACE									
						13	65.29		
						14	64.29		
						15	63.29		
						16	62.29		
						17	61.29		
						18	60.29		
						19	59.29		
						20	58.29		
						21	57.29		
						22	56.29		
						23	55.29		
23.95									
End of Borehole Practical DCPT refusal at 23.95m depth (GWL @ 1.30m - Aug. 8, 2018)									



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

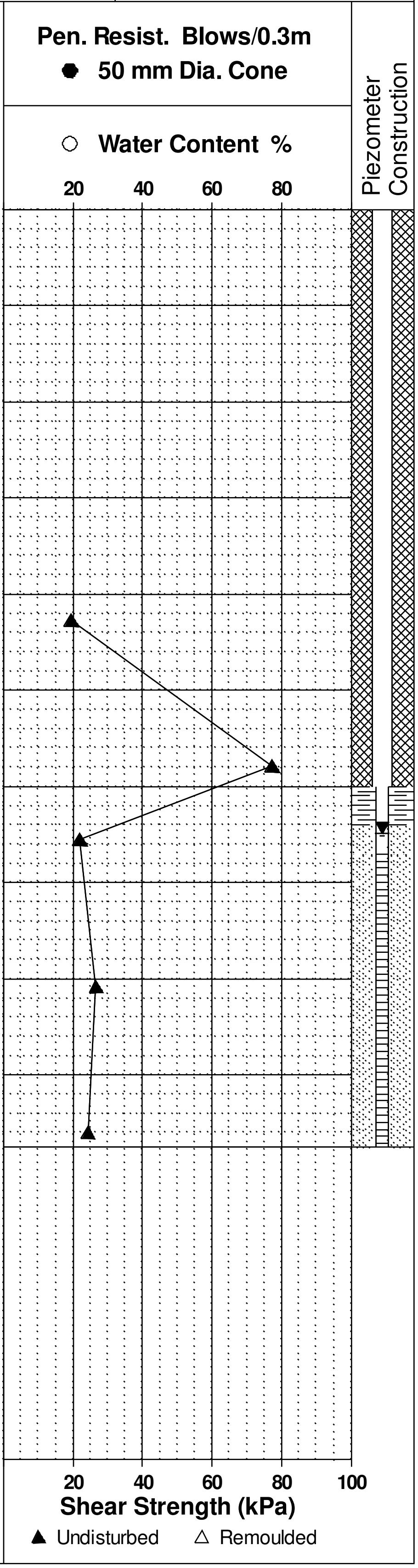
DATE 3 August 2018

FILE NO. PG4592

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Brown silty clay with sand, gravel, cobbles		AU	1		
		SS	2	50	17
		SS	3	0	7
2.59		SS	4	25	3
Very loose, brown **SILTY SAND**		SS	5	100	3
3.50					
Soft to firm, grey **SILTY CLAY**		TW	6	100	
9.75					
End of Borehole					
(GWL @ 6.49m - Aug. 8, 2018)					

DEPTH (m)	ELEV. (m)
0	78.67
1	77.67
2	76.67
3	75.67
4	74.67
5	73.67
6	72.67
7	71.67
8	70.67
9	69.67



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 3 August 2018

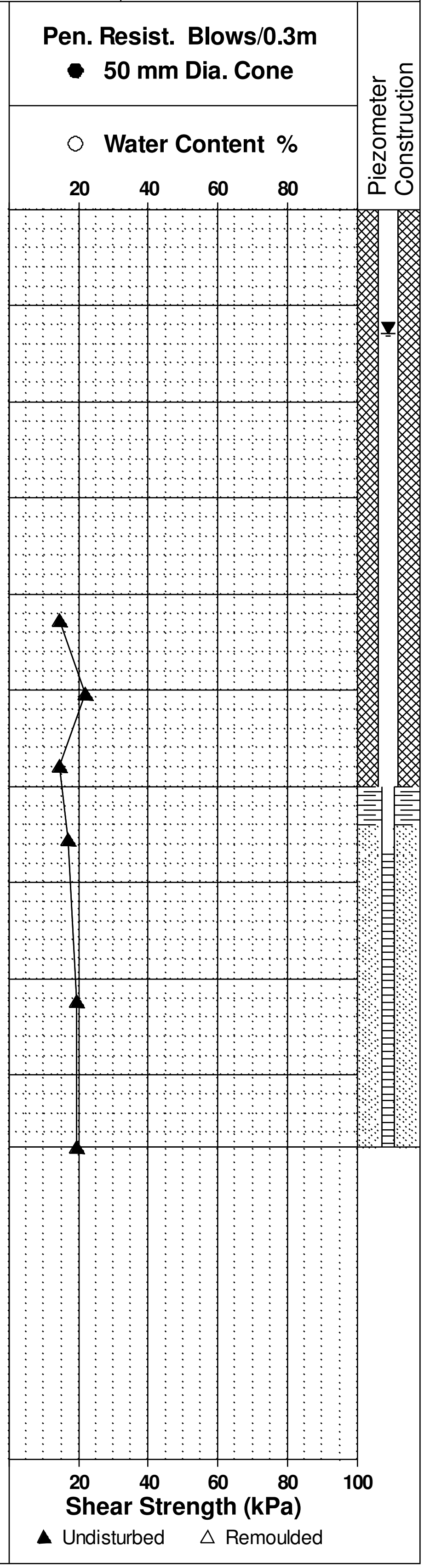
FILE NO. PG4592

HOLE NO. BH 3

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
TOPSOIL	0.05
FILL: Brown silty clay with sand	1.83
Compact, brown **SAND**	2.44
Soft, grey **SILTY CLAY**	9.75
End of Borehole	
(GWL @ 1.29m - Aug. 8, 2018)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	50	9
SS	3	50	17
SS	4	100	W
TW	5	100	
TW	6	100	

DEPTH (m)	ELEV. (m)
0	77.67
1	76.67
2	75.67
3	74.67
4	73.67
5	72.67
6	71.67
7	70.67
8	69.67
9	68.67



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

FILE NO. **PG4592**

REMARKS

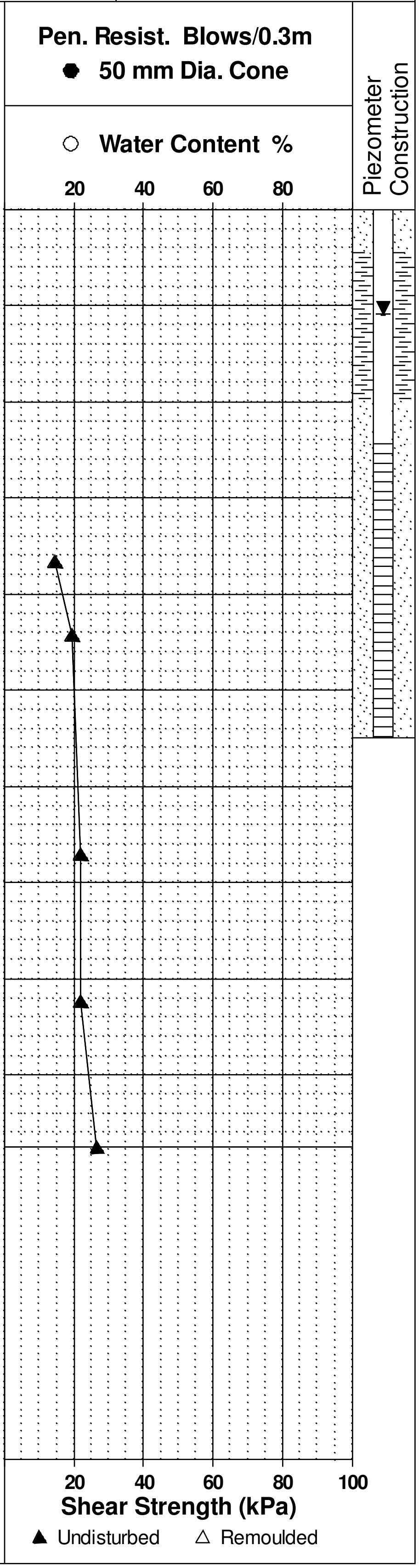
HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger

DATE 3 August 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD
GROUND SURFACE					
FILL: Gravelly sand					
0.60					
FILL: Grey silty clay with sand - some gravel by 1.5m depth		SS	1	50	5
		SS	2	58	10
		SS	3	42	2
3.05					
Soft to firm, brown to grey **SILTY CLAY**		SS	4	100	2
		TW	5	100	
9.75					
End of Borehole (GWL @ 1.08m - Aug. 8, 2018)					

DEPTH (m)	ELEV. (m)
0	77.98
1	76.98
2	75.98
3	74.98
4	73.98
5	72.98
6	71.98
7	70.98
8	69.98
9	68.98



patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 1

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	77.87	
						1	76.87	
FILL: Silty sand topsoil with organics, trace gravel, cobbles and boulder	▨	G	1					
						2	75.87	
		G	2					
3.10						3	74.87	▽
Brown **SILTY SAND**, trace organics		G	3					
3.50								
End of Test Pit (Open hole GWL @ 3.1 m depth)								

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

○ **Water Content %**

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

DATUM Ground surface elevation were provided by Annis O'Sullivan Vollebekk Ltd.

REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	77.56	
TOPSOIL 0.08								
		G	1					∇
FILL: Brown silty sand, some gravel, cobbles, boulders, asphalt, concrete, foam, construction debris, metal, plaster, pipe, brick and clay						1	76.56	
		G	2			2	75.56	
3.20						3	74.56	
Brown **SILTY SAND**, trace organics 3.50		G	3					
End of Test Pit								
(Open hole GWL @ 0.6 m depth)								

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

○ **Water Content %**

20 40 60 80

20 40 60 80 100
Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
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REMARKS

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DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 3

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
GROUND SURFACE						0	77.51	
						1	76.51	
FILL: Brown silty sand, trace brown silty clay, some sand, gravel and cobbles, trace asphalt, brick, organics and concrete.		G	1					
		G	2					
						2	75.51	
2.30								
Brown **SILTY SAND**, trace organics		G	3					
2.90								
Red **SILTY CLAY** -grey by 3.3 m depth		G	4			3	74.51	
								▽
		G	5					
3.50								
End of Test Pit (Open hole GWL @ 3.3 m depth)								

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

○ **Water Content %**
20 40 60 80

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	77.66		
		G	1						
									▽
						1	76.66		
FILL: Brown silty sand, some asphalt, gravel, cobbles, boulders, construction debris and brick									
		G	2						
						2	75.66		
		G	3						
						3	74.66		
3.10 Brown **SILTY SAND**		G	4						
3.30 Red **SILTY CLAY**		G	5						
3.50 End of Test Pit (Open hole GWL @ 0.75 m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

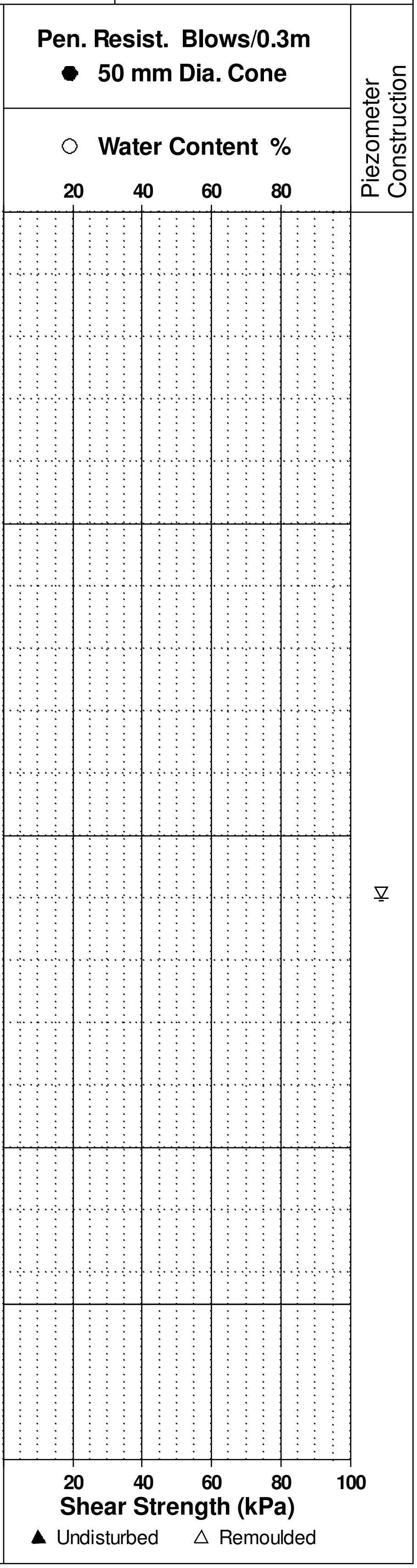
BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	77.93
	▨	G	1				
FILL: Brown silty sand to brown silty clay, some sand, gravel, cobbles and boulders, trace organics, brick, glass, tile, and construction debris	▨					1	76.93
	▨	G	2				
	▨					2	75.93
2.20							
Brown **SILTY SAND**	⁝	G	3				
3.00						3	74.93
Red **SILTY CLAY** -grey by 3.3 m depth	▧	G	4				
	▧	G	5				
3.50							
End of Test Pit (Open hole GWL @ 2.2 m depth)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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Ottawa, Ontario

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REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD				
FILL: Brown silty clay, some sand, gravel, cobbles, trace organics and construction debris						0	78.45		
		G	1						
						1	77.45		
						2	76.45		
2.30									
Brown **SILTY SAND**		G	2						
2.50									
Red **SILTY CLAY**		G	3						
						3	75.45		∇
3.50									
End of Test Pit (Open hole GWL @ 3.0 m depth)									

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
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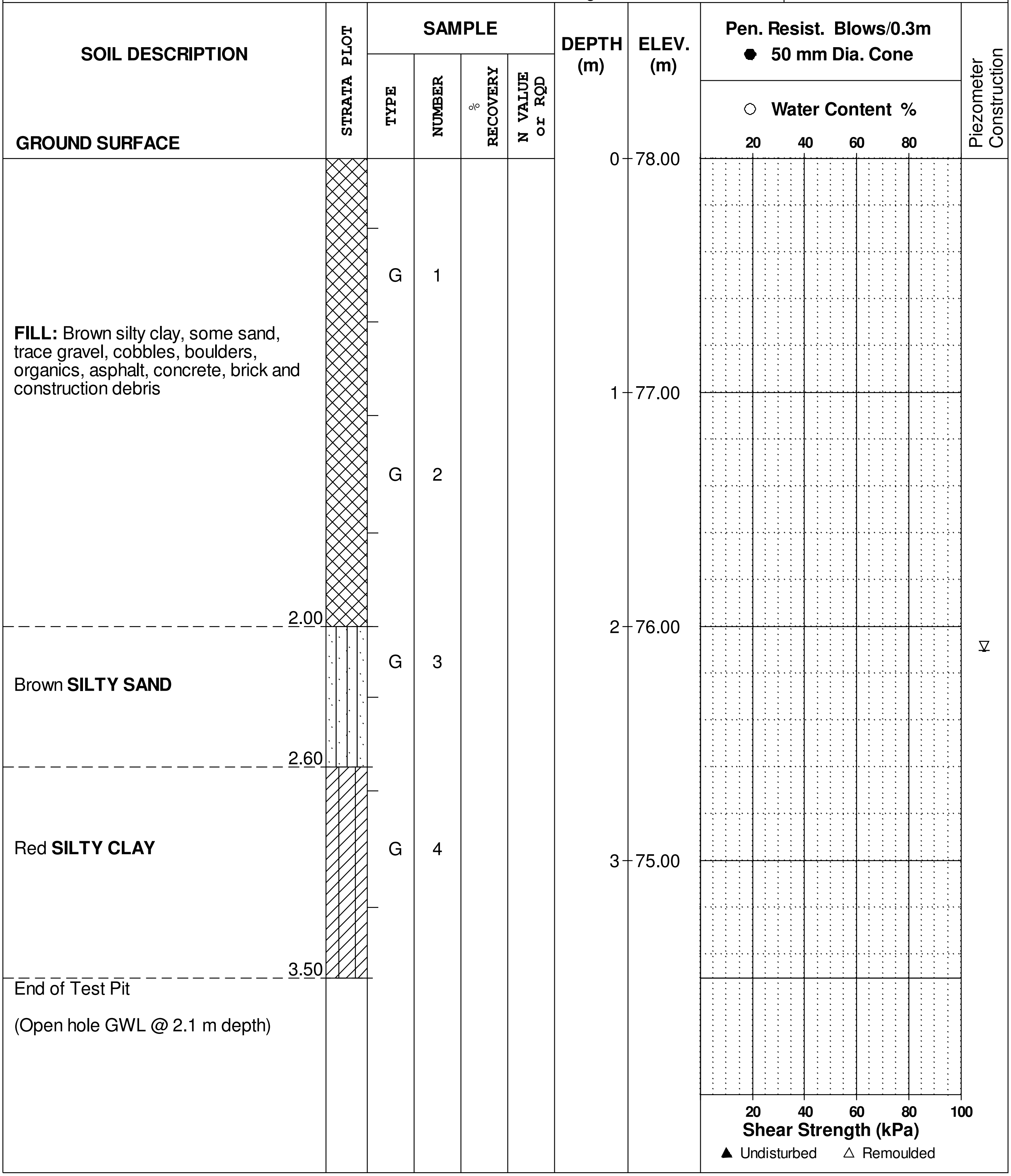
REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 7



SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	78.00		
FILL: Brown silty clay, some sand, trace gravel, cobbles, boulders, organics, asphalt, concrete, brick and construction debris		G	1						
						1	77.00		
		G	2						
2.00						2	76.00		
Brown **SILTY SAND**		G	3						∇
2.60									
Red **SILTY CLAY**		G	4			3	75.00		
3.50									
End of Test Pit (Open hole GWL @ 2.1 m depth)									

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SOIL PROFILE AND TEST DATA

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REMARKS

BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone ○ Water Content % 20 40 60 80	Piezometer Construction
GROUND SURFACE						0	78.00		
FILL: Brown silty clay, some sand, gravel, cobbles, trace boulders, organics, asphalt, brick and construction debris	☒	G	1						
						1	77.00		
		G	2						
						2	76.00		
2.05									
Red **SILTY CLAY**	▨	G	3						
2.55									
End of Test Pit (TP dry upon completion)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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REMARKS

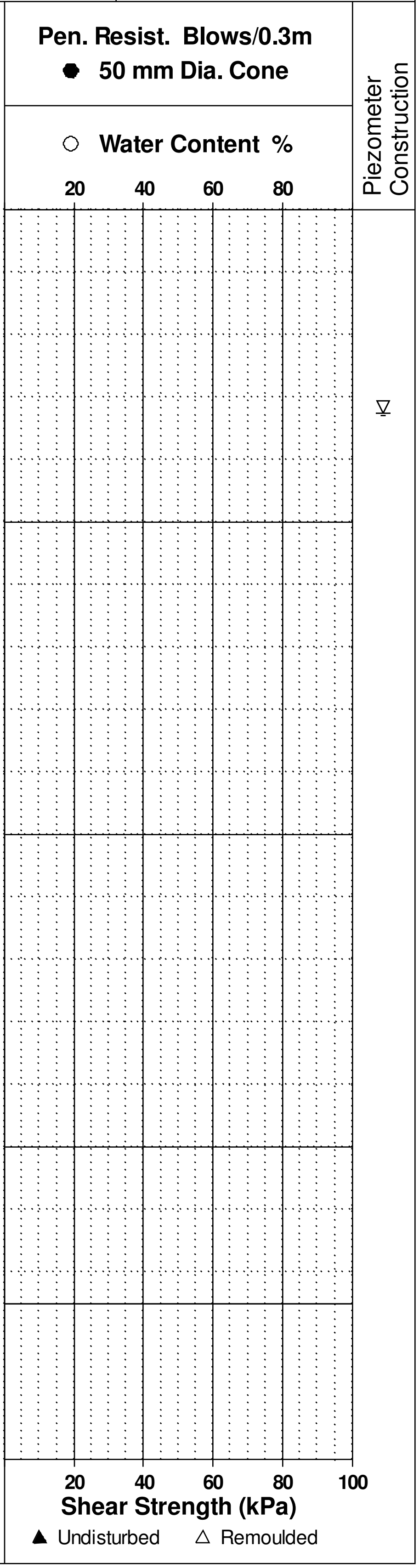
BORINGS BY Backhoe

DATE 1 August 2018

FILE NO. PG4592

HOLE NO. TP 9

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.17
FILL: Brown silty sand, some clay, gravel, cobbles, boulders, and construction debris	▨	G	1				
						1	77.17
		G	2				
						2	76.17
		G	3				
						3	75.17
3.50		G	4				
End of Test Pit (Open hole GWL @ 0.65 m depth)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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FILE NO. PG4592

REMARKS

HOLE NO. TP10

BORINGS BY Backhoe

DATE 1 August 2018

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	78.27		
		G	1						
FILL: Brown silty sand, some gravel and cobbles, trace boulders, brick and organics						1	77.27		
		G	2						
						2	76.27		
2.10									
Brown **SILTY SAND**		G	3						
2.30									
Red **SILTY CLAY**		G	4						
2.60									
End of Test Pit (TP dry upon completion)									

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
Ottawa, Ontario

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FILE NO. **PG4592**

REMARKS

HOLE NO. **TP11**

BORINGS BY Backhoe **DATE** 1 August 2018

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			20 40 60 80	
GROUND SURFACE						0	78.07		
	▩	G	1						
FILL: Brown silty sand, some topsoil, trace organics, gravel, cobbles, boulders and clay	▩					1	77.07		
	▩	G	2						
2.05	▩					2	76.07		
Brown **SILTY SAND** 2.25	░	G	3						
Red **SILTY CLAY**	▨	G	4						
-grey by 3.1 m depth	▨	G	5			3	75.07		∇
3.50	▨								
End of Test Pit (Open hole GWL @ 3.1 m depth)									

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
5510 Boundary Road
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FILE NO. PG4592

REMARKS

HOLE NO. TP12

BORINGS BY Backhoe

DATE 1 August 2018

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	76.25
FILL: Brown silty sand, some clay, trace gravel, cobbles, boulders and organics	G	1			1	75.25
1.40						
Brown **SILTY SAND**	G	2			2	74.25
2.10						
Red **SILTY CLAY** -grey by 2.6 m depth	G	3				
	G	4			3	73.25
3.50						
End of Test Pit (Open hole GWL @ 2.6 m depth)						

STRATA PLOT

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

○ Water Content %

20 40 60 80

Piezometer Construction

▽

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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 strength of cohesionless soils is the relative density, 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.

Relative Density	‘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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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 is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC% - Natural moisture 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 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 x 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)

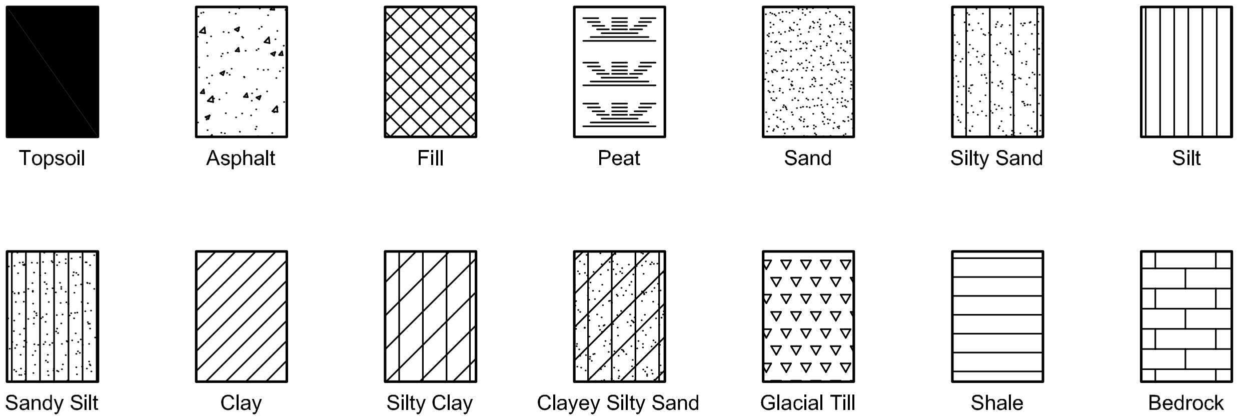
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

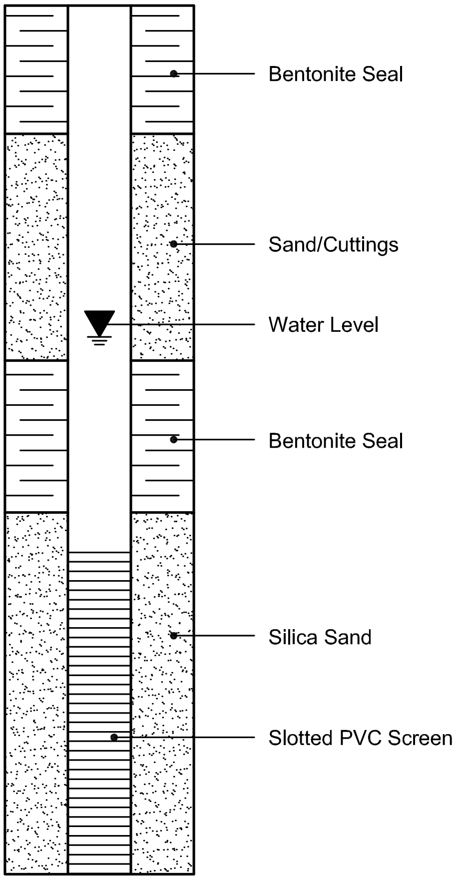
SYMBOLS AND TERMS (continued)

STRATA PLOT

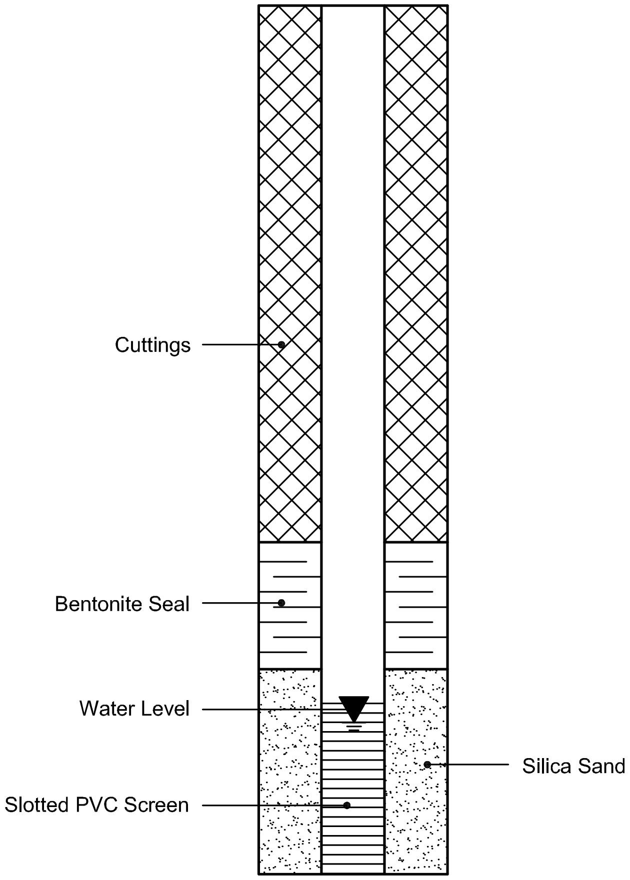


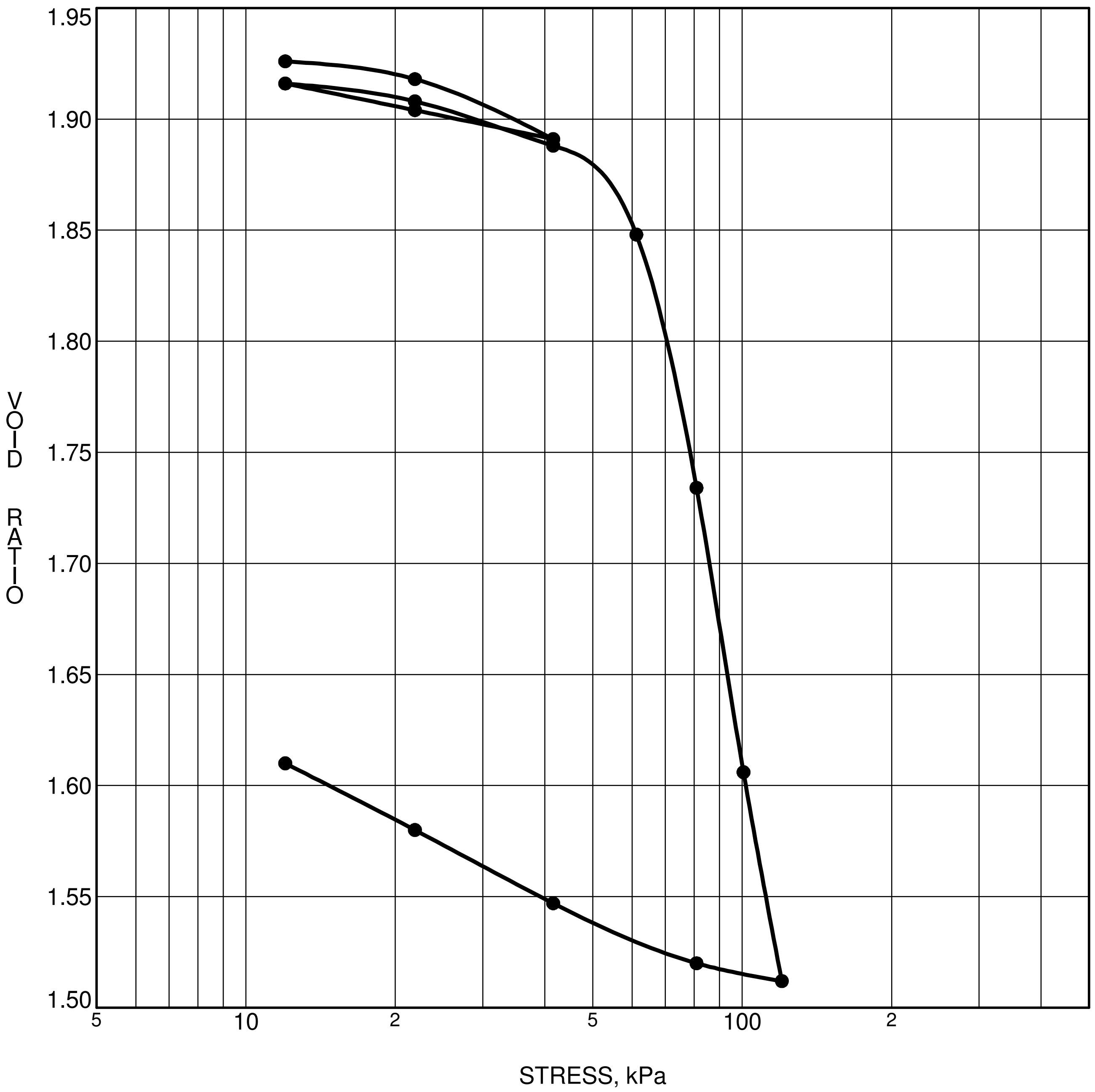
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





CONSOLIDATION TEST DATA SUMMARY

Borehole No.	**BH 2**	p'_o	**27.8** kPa	Ccr	**0.047**
Sample No.	**TW 6**	p'_c	**64.66** kPa	Cc	**1.293**
Sample Depth	**5.05** m	OC Ratio	**2.3**	Wo	**70.6** %
Sample Elev.	**73.62** m	Void Ratio	**1.941**	Unit Wt.	**15.6** kN/m^3

CLIENT **Day and Ross**

PROJECT **Geotechnical Investigation - 5510 Boundary Road**

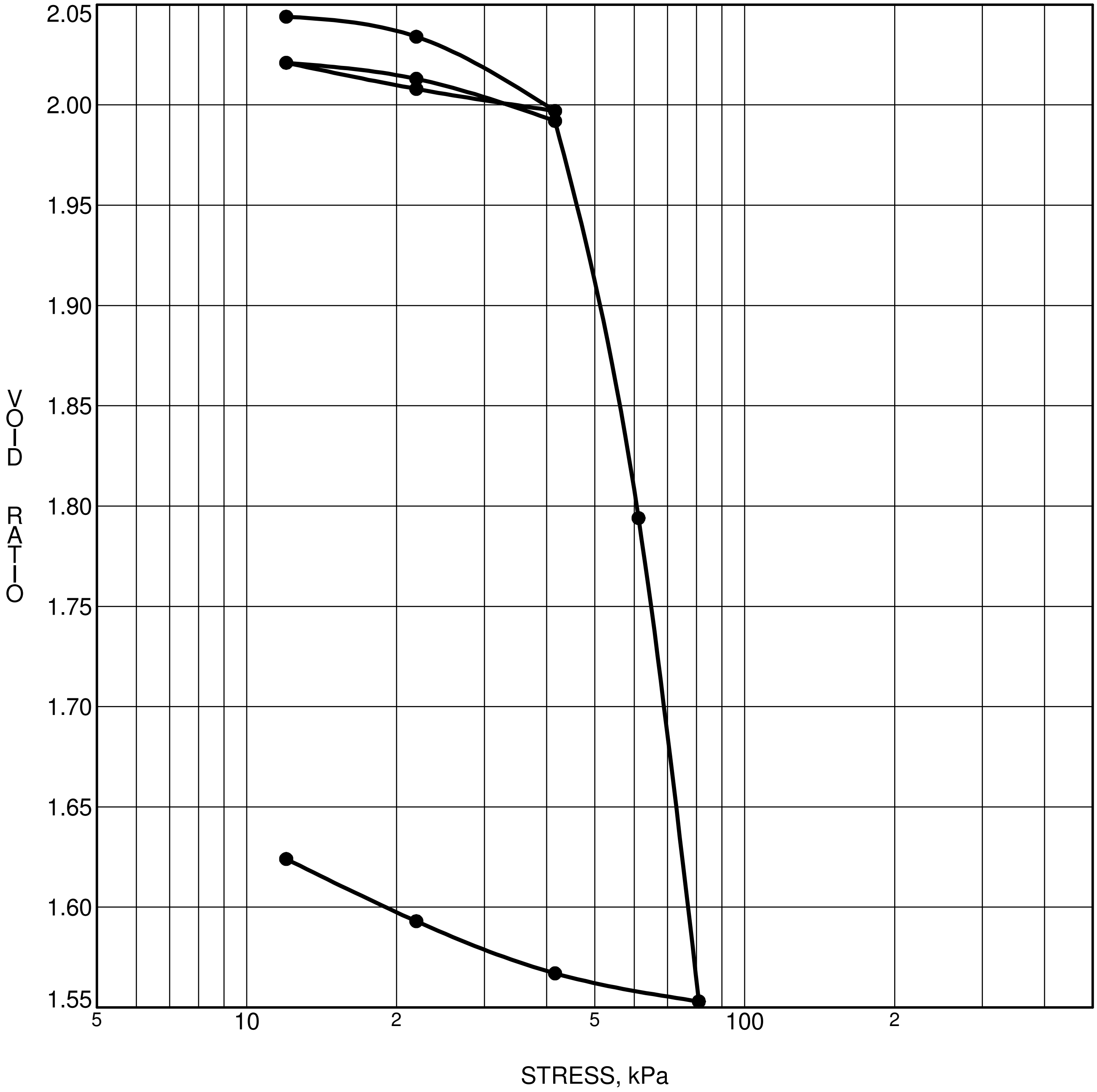
FILE NO. **PG4592**

DATE **30/08/2018**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY					
Borehole No.	**BH 3**	p'o	**16.32** kPa	Ccr	**0.048**
Sample No.	**TW 5**	p'c	**50** kPa	Cc	**1.986**
Sample Depth	**3.38** m	OC Ratio	**3.1**	Wo	**75.0** %
Sample Elev.	**74.29** m	Void Ratio	**2.062**	Unit Wt.	**15.4** kN/m 3

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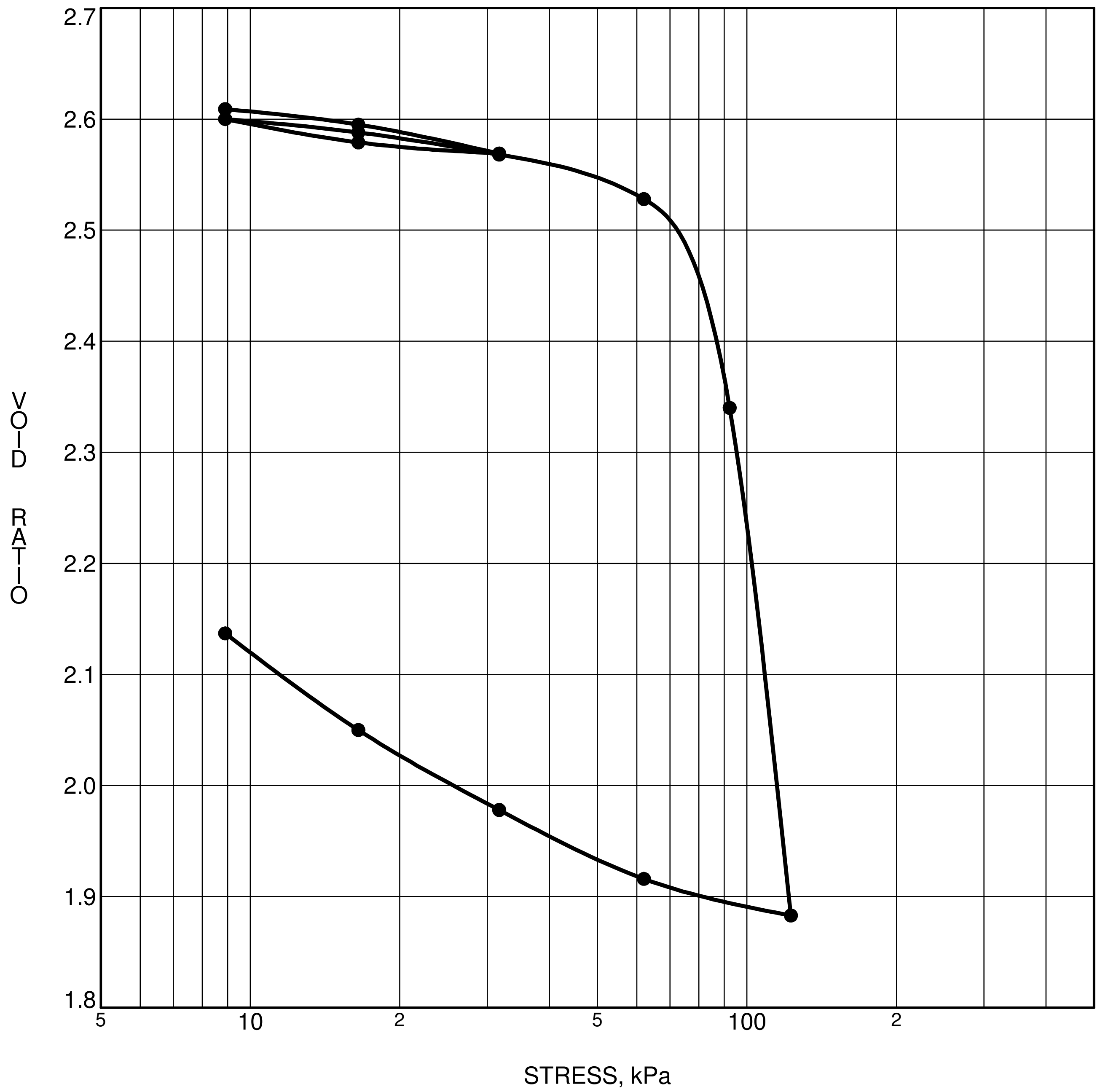
CONSOLIDATION TEST



CONSOLIDATION TEST DATA SUMMARY								
Borehole No.	**BH 4**		p'o	**31.82**	kPa	Ccr	**0.058**	
Sample No.	**TW 5**		p'c	**82.79**	kPa	Cc	**3.722**	
Sample Depth	**5.69**	m	OC Ratio	**2.6**		Wo	**94.9**	%
Sample Elev.	**72.29**	m	Void Ratio	**2.61**		Unit Wt.	**14.6**	kN/m^3

CLIENT **Day and Ross**

PROJECT **Geotechnical Investigation - 5510 Boundary Road**

FILE NO. **PG4592**

DATE **30/08/2018**

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CONSOLIDATION TEST

APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – SHEAR WAVE VELOCITY PROFILE AT SHOT LOCATION 34.5 m

DRAWING PG4592 - 1 – TEST HOLE LOCATION PLAN

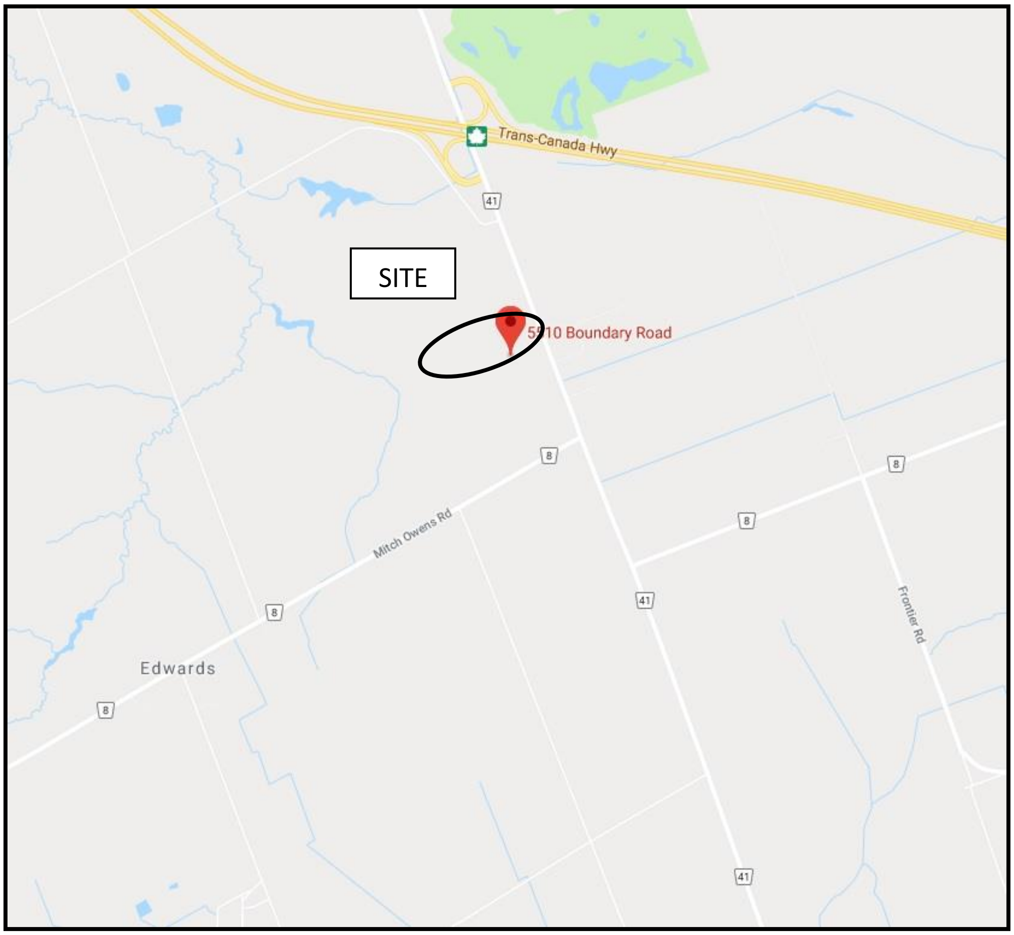


FIGURE 1

KEY PLAN

PATERSON
GROUP

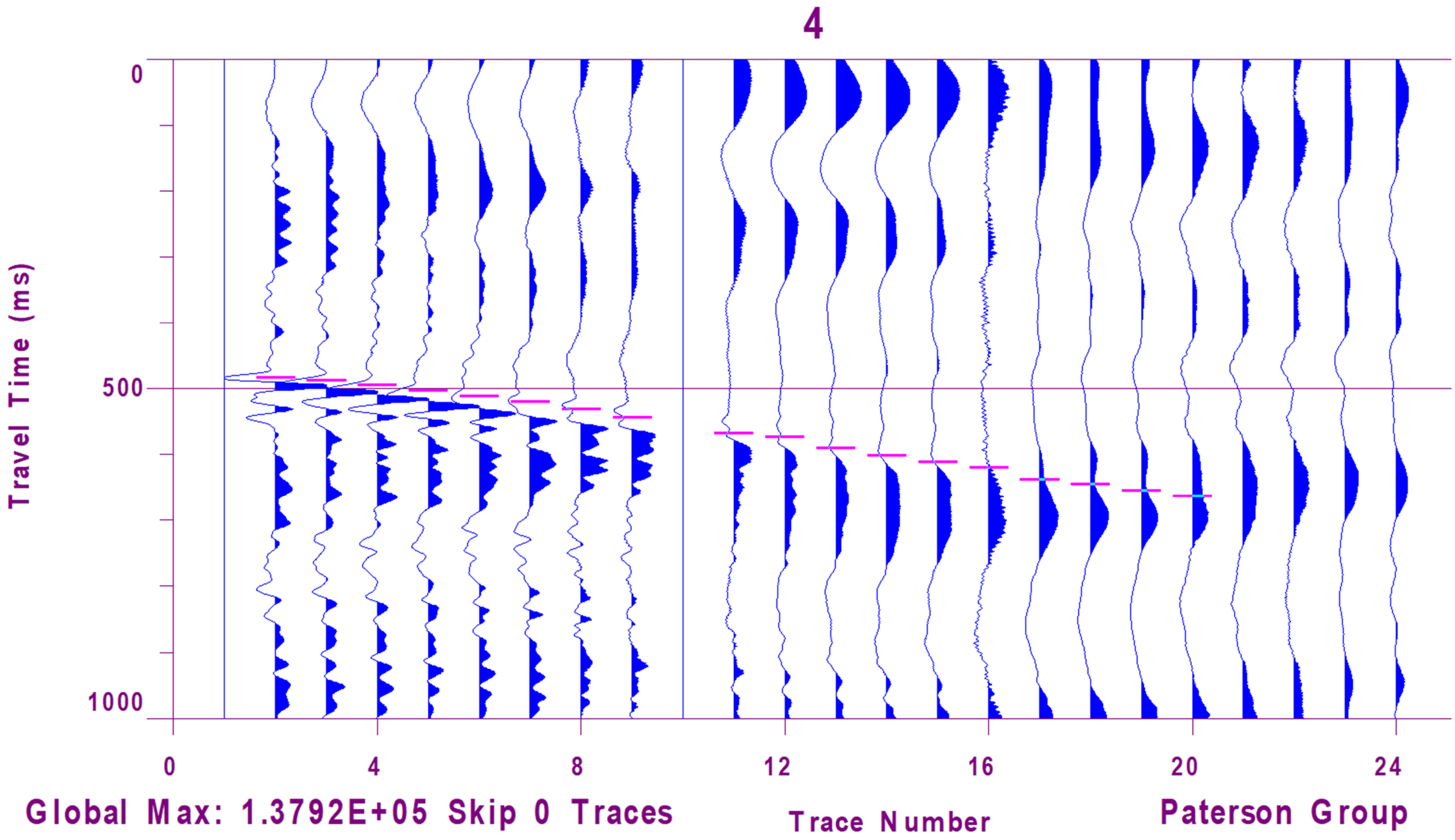


Figure 2 – Shear Wave Velocity Profile at Shot Location -30 m

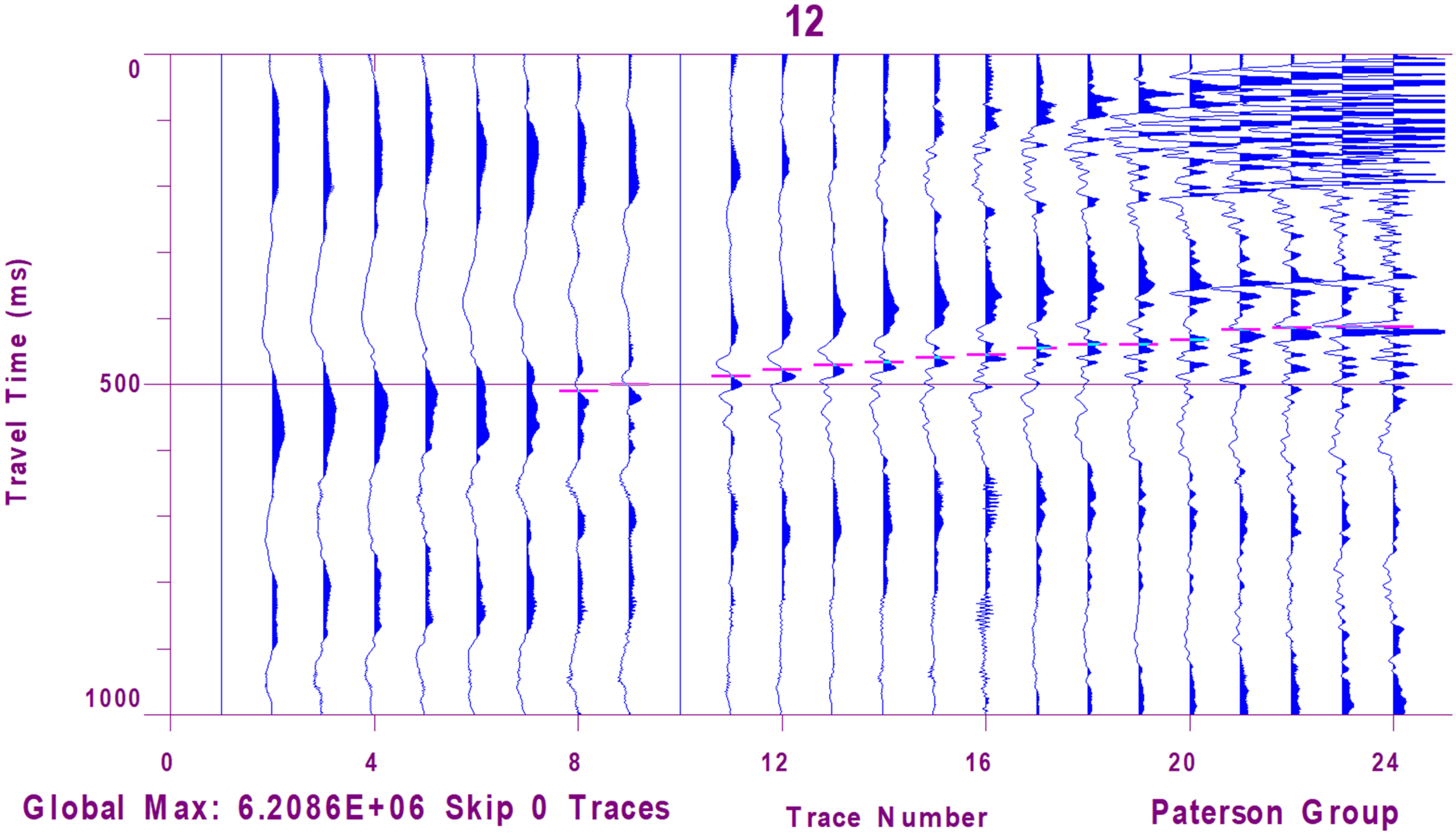
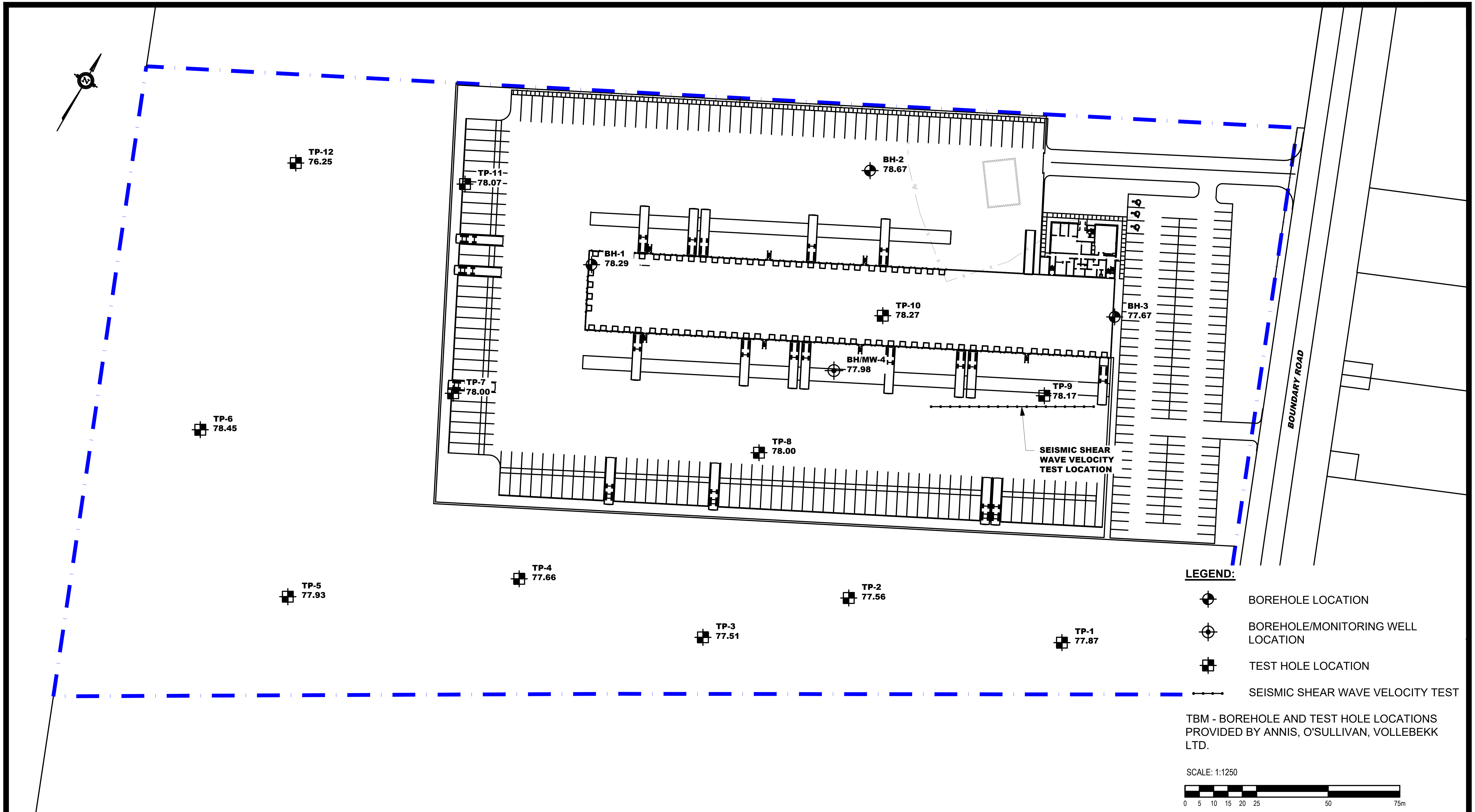


Figure 3 – Shear Wave Velocity Profile at Shot Location 72 m



PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
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NO.	REVISIONS	DATE	INITIAL

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GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE COMPLEX - 5510 BOUNDARY ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:1250
Drawn by: GR
Checked by: FAS
Approved by: DG
Date: 08/2018
Report No.: PG4592-1
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