Geotechnical Investigation

Proposed Residential Development

Chapman Mills Drive and Riocan Avenue
Ottawa, Ontario

Prepared for Minto Communities

Report PG5636-2 Revision 2, dated June 27, 2023

Table of Contents

Appendices

Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Atterberg Limit's Testing Results
Analytical Test Results

Appendix 2 Figure 1 - Key Plan
Figure 2 and Figure 3 – Shear Wave Velocity Testing Profiles
Figure 4 - Aerial Photograph – 1999
Figure 5 - Aerial Photograph – 2011
Figure 6 - Aerial Photograph – 2017
Drawing PG5636-1 - Test Hole Location Plan
Drawing PG5636-2 – Permissible Grade Raise Plan
Drawing PG5636-3 – Tree Planting Setback Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Minto Communities to conduct a geotechnical investigation for the proposed residential development to be located southeast of the intersection of Chapman Mills Drive and Riocan Avenue in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the geotechnical investigation was to:

- Determine the subsoil and groundwater conditions at this site by means of a test hole program.

- Provide geotechnical recommendations pertaining to 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

It is understood that the proposed development will consist of low-rise back-to-back style townhouse residential dwellings with one level of underground parking that will be located below the majority of the centrally located blocks. Associated driveways, local roadways and landscaped areas are also anticipated as part of the development.

It is expected that the proposed residential development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on August 22 and August 23, 2011, and consisted of advancing a total of ten (10) boreholes to a maximum depth of 12.2 m below the existing ground surface. An additional investigation had been undertaken by Paterson throughout the remainder of the property parcel of 3265 Jockvale Road during January of 2021 to supplement the findings throughout the larger property parcel and the subject site.

The test holes were completed using a low clearance drill rig operated by a two-person crew. The drilling procedure consisted of drilling to the required depths at the selected locations, and sampling and testing the overburden. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using 47.6 mm inside diameter coring equipment.

All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

The 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.

Overburden thickness was also evaluated during the course of the previous investigations by dynamic cone penetration testing (DCPT) at BH 3 and BH 5.

The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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.

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

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

Flexible polyethylene standpipes were installed within the boreholes to measure the stabilized groundwater levels subsequent to completion of the sampling program. The groundwater level measurements are shown on the Soil Profile and Test Data sheets and are tabulated in Subsection 4.3**.**

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG5636-1 - 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. Two samples were submitted for Atterberg Limit’s testing and the results are discussed in Subsection 4.2.

3.4 Analytical Testing

One (1) 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 samples. 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 consists of currently undeveloped former agricultural land. The existing ground surface ranges between 101 and 95 m and gradually slopes downward from north to south and is approximately at grade with adjacent properties and roadways.

Based on available aerial photographs, topographic surveys taken of the subject site and observations made by Paterson field personnel at the time of the current and supplemental investigations it has been observed that portions of the subject site have undergone historical soil movement activities. Reference should be made to Figure 4 - Aerial Photograph - 1999, Figure 5 - Aerial Photograph - 2011 and Figure 6 - Aerial Photograph - 2017 which illustrate the general presence and movement of fill throughout the subject site.

The subject site is bordered to the north by Glenroy Gilbert Drive and further by a commercial plaza and two to three-storey townhouse and condo dwellings, to the west and south by undeveloped land and to the east by Longfields Drive.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of either fill or topsoil underlain by a silty clay crust and/or a glacial till deposit. Where encountered, the existing fill layer was observed to range between 0.3 to 0.7 m in depth.

The surficial layer of topsoil and/or fill was observed to be underlain by a weathered crust layer of silty clay at the locations of boreholes BH 2, BH 2A, BH 3 and BH 4. This crust layer was observed to range between 0.9 and 2.2 m in thickness.

Glacial till was observed underlying the above-noted deposits at all test hole locations. The fine-matrix of the glacial till generally consisted of silty sand with varying amounts of clay. A significant amount of cobbles, boulders and oversized boulders are also present throughout the glacial till deposit encountered throughout the subject site.

Practical refusal to augering was encountered at an approximate depth of 2.4 m, 7.0 m, 3.8 m and 6.4 m below existing ground surface at the location of boreholes BH 2, BH 6, BH 7 and BH 7A, respectively. Practical refusal to DCPT was encountered at an approximate depth of 12.1 m and 11.5 m below existing ground surface at boreholes BH 3 and BH 5.

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.

Atterberg Limits Testing

Atterberg limits testing was completed on silty clay samples recovered from BH 6-21 and BH 7-21. The result of the Atterberg limits tests is presented in Table 1 and on the Atterberg Limits Testing Results sheet in Appendix 1.

Table 1 - Atterberg Limits Results

Sample	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
BH 6-21 SS3	1.83	55	29	22	MH
BH 7-21 SS4	2.59	72	35	37	MH

Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plasticity Index; w: water content;
MH: Inorganic silt of high plasticity

Bedrock

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded sandstone and dolomite of the March formation with a drift thickness between 10 to 15 m.

4.3 Groundwater

Groundwater levels were measured during the current investigation on August 30, 2011, within the installed piezometers. The groundwater level measurements are presented in Table 1 below.

Table 1 – Summary of Groundwater Levels				
PG5636 – Current Investigation				
Test Hole Number	**Ground Surface Elevation (m)**	**Measured Groundwater Level / Groundwater Infiltration for Boreholes**		**Dated Recorded**
		Depth (m)	**Elevation (m)**	
BH 1	99.83	5.60	94.23	August 30, 2011
BH 2A	97.02	3.95	93.07	
BH 3	97.23	4.34	92.89	
BH 4	95.23	2.94	92.29	
BH 5	101.62	Destroyed	n/a	
BH 6	99.95	Dry	n/a	
BH 7A	100.25	4.25	96.00	
BH 8	96.75	3.12	93.63	
Note: The ground surface elevation at each borehole location for the current investigation was surveyed using a handheld GPS referenced to a geodetic datum.				

The remainder of the boreholes appeared to be dry upon completion of sampling and were subsequently backfilled. Long-term groundwater levels can also be estimated based on recovered soils samples moisture levels, soil sample coloring and consistency. Based on this methodology, the long-term groundwater level is estimated to be located at a **4 to 5 m** depth below ground surface.

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

From a geotechnical perspective, the subject site is considered suitable for the proposed development. Foundations for the proposed residential dwellings may be founded on conventional spread footings placed on an undisturbed, very stiff brown silty clay and/or dense to compact glacial till deposit. Further, the proposed underground parking level may be founded on conventional spread footings placed on an undisturbed, dense to compact glacial till deposit.

It is anticipated that cobbles and boulders will be encountered frequently throughout servicing trenches and building excavations. All contractors should be prepared for the removal of boulders and potentially oversized boulders throughout the subject site.

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

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, and any deleterious fill, such as those containing organic materials, should be stripped from under any buildings 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.

The existing fill, where free of organics and deleterious materials, can be left in place below the proposed floor slab and beyond the lateral support zones for footings. If considered to be left in place as subgrade, it is recommended that the existing fill be proof-rolled **under dry conditions and above freezing temperatures** by an adequately sized vibratory roller making several passes to achieve optimum compaction levels.

The compaction program should be reviewed and approved by Paterson personnel at the time of construction. In poor performing areas, the unsuitable portions of the existing fill should be removed and reinstated with an approved engineered fill, such as OPSS Granular B Type II.

Fill Placement

Fill used for grading beneath the proposed buildings and structures should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill should be tested and approved prior to delivery to the site. It should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Engineered fill placed beneath the buildings and structures should be compacted to at least 98% of its 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. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids.

If site-excavated workable, brown, stiff silty clay or glacial till, free of organics and deleterious materials and oversized boulders, are considered to be used to build up the subgrade level for areas to be paved, between footings in the underground parking levels, or to backfill around the proposed buildings, it is recommended that the material be placed under dry conditions and above freezing temperatures. The site-excavated soils should be compacted in maximum 300 mm thick lifts to at least 95% of the material's SPMDD using a vibratory sheepsfoot roller. The placement of the material should be reviewed and approved by Paterson at the time of construction.

All stones and cobbles larger than 200 mm in their longest dimension should be segregated from stockpiles prior to re-use. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Conventional Shallow Foundations

Strip footings, up to 3 m wide, and pad footings, up to 5 m wide, placed on an undisturbed, very stiff silty clay bearing surface can be designed using a bearing resistance value at serviceability limit state (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit state (ULS) of **225 kPa**.

Conventional spread footings placed on an undisturbed, compact to very dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit state (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**.

An undisturbed soil bearing surface consists of a surface from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in dry conditions, prior to the placement of concrete for footings.

Strip footings placed on a soil bearing surface and designed using the bearing resistance values at SLS given above will be subjected to potential post construction total and differential settlements of 25 and 20 mm, respectively.

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 overburden 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.

Permissible Grade Raise

Due to the presence of the underlying silty clay layer within the southern portion of the subject site, a permissible grade raise restriction of **2.0 m** for the area outlined on Drawing PG5636-2 - Permissible Grade Raise Areas in Appendix 2. However, it is expected that the proposed underground parking structure will require the removal of the shallow silty clay deposit and therefore, a permissible grade raise restriction is most likely not applicable for this structure. Footings bearing on a compact to dense glacial till bearing surface will not be subjected to a permissible grade raise restriction.

If higher than permissible grade raises are required, preloading with or without a surcharge, lightweight fill, and/or other measures should be investigated to reduce the risks of unacceptable long-term post construction total and differential settlements.

5.4 Design for Earthquakes

Shear wave velocity testing was completed for a portion of the subject site as part of the 2011 investigation 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 (OBC) 2012. The results of the shear wave velocity testing are attached to the present report.

Field Program

The shear wave testing location is presented in Drawing PG5636-1 - Test Hole Location Plan attached to this report. Paterson field personnel placed 24 horizontal geophones in a straight line roughly in line with the east boundary line, along Longfields Drive.

The 4.5 Hz. horizontal geophones were 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 are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at the centre of the geophone array and 3, 4.5 and 30 m away from the first and last geophone.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were 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, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation. 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.

The seismic survey did not trace a refraction head-wave from the underlying bedrock due to the bouldery glacial till deposit. The bedrock shear wave velocity was estimated based on the results of seismic surveys completed on the same deposit with bedrock of the same or higher degree of weathering.

The bedrock shear wave velocity can be taken conservatively to be 1,600 m/s and the overburden soil shear wave velocities were calculated to be 236 m/s and 470 m/s for the silty clay and glacial till layers, respectively. Based on available geological mapping, the bedrock is expected at a 25 m depth in a worst-case scenario.

Townhouses Founded on Silty Clay Overburden

For conventional spread footings bearing directly on silty clay, the Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below.

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

$$V_{s30=} \frac{30\,m}{\left(\frac{2\,m}{236\,m/s} + \frac{23\,m}{470\,m/s} + \frac{5\,m}{1{,}600\,m/s}\right)}$$

$$V_{s30=}\,495\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for footings supported on silty clay is 495 m/s. Therefore, a **Site Class C** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are no susceptible to liquefaction.

Parking Garage Footings Supported on Glacial Till

For conventional spread footings associated with the parking garage bearing on the dense glacial till, the Vs_{30} was calculated using the standard equation for average shear wave velocity provided in the OBC 2012 and as presented below.

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

$$V_{s30=} \frac{30\ m}{\left(\frac{25\ m}{470\ m/s} + \frac{5\ m}{1{,}600\ m/s}\right)}$$

$$V_{s30=}\ 532\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, Vs_{30}, for the parking garage footings supported on glacial till is 532 m/s. Therefore, a **Site Class C** is applicable for design of the proposed building, as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are no susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious material, containing organic matter, within the footprints of the proposed buildings, the approved native soil surface will be considered to be an acceptable subgrade surface on which to commence backfilling for floor slab construction.

The recommended pavement structures noted in Subsection 5.7 will be applicable for the founding level of the proposed parking garage structure. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consists of 19 mm clear crushed stone.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular B Type II, with a maximum particle size of 50 mm, is recommended for backfilling soft spots below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the clear stone under the lower parking level. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed (discussed further in Subsection 6.1).

5.6 Basement Wall

Where the soil is to be retained, there are several combinations of backfill materials and retaining soils for the basement walls of the subject structure. However, the conditions should be designed by assuming the retaining soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. The applicable effective unit weight of the retained soil can be estimated as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

The total earth pressure (P_{AE}) includes both the static earth pressure component (P_o) and the seismic component (DP_{AE}).

Lateral Earth Pressures

The static horizontal earth pressure (P_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil (0.5)
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $Ko \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32 g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Design

Podium Deck Area

It is anticipated that the podium deck structure will be provided car only parking areas, access lanes, fire truck lanes and loading areas. Based on the concrete slab subgrade, the following pavement structures may be considered for design purposes:

Table 2 - Recommended Pavement Structure - Car Only Parking Areas (Podium Deck)

Thickness (mm)	Material Description
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
200**	**BASE** – OPSS Granular B Type II
See Below*	**Thermal Break*** - Rigid Insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**

SUBGRADE – Reinforced Concrete Podium Deck
*If specified by others, not required from a geotechnical perspective
**Thickness is dependant on grade of insulation as noted in preceding paragraph

Table 3 - Recommended Pavement Structure - Recommended Pavement Structure Access Lanes, Fire-Truck Lane, Ramp 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
300*	**BASE** – OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid Insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and IKO Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck *If specified by others, not required from a geotechnical perspective **Thickness is dependant on grade of insulation as noted in preceding paragraph	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition a 5H:1V is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 600 mm below the top of the podium slab a minimum of 1.5 m from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High-Load 100 (HI-100), High-Load 60 (HI-60) or High Load 40 (HI-40). The pavement structures base layer thickness in Table 2 and Table 3 may be reduced by 25 mm if HI-100 is considered for this project. It should be noted that SM (styrofoam) rigid insulation is not considered suitable for this application.

Pavement Structure Over Overburden

Beyond the podium deck, the following pavement structures may be considered for driveways, local residential roads, and heavy-duty access lanes and loading areas for design purposes.

Table 4 - Recommended Pavement Structure – Driveways and Fire Truck Lanes	
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 fill	

Table 5 - Recommended Pavement Structure – Local Residential Roadways, 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
400	**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 fill.	

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. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

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 vibratory equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Due to the impervious nature of the silty clay deposit, where it is anticipated to be encountered at the pavement structures subgrade level consideration should be given to installing subdrains during the pavement construction. These drains should be installed at each catch basin, be at least 3 m long and should extend in four orthogonal directions or longitudinally when placed along a curb. 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 residential structures and underground parking level foundation. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded by 150 mm of 10 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 or dedicated sump pump systems.

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 Miradrain G100N or 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.

Backfill material below sidewalk subgrade areas or other settlement sensitive structures should consist of free draining, non-frost susceptible material placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Underfloor Drainage

In order to divert surface water collected by the buildings foundation drainage system to the buildings sump system within the underground parking level, it is recommended that an underfloor drainage system be implemented. For design purposes, the underfloor drainage pipes should consist of a 150 mm diameter corrugated perforated pipe surrounded by a geosock and a minimum of 150 mm of 19 mm clear crushed stone on all of its sides. The underfloor drainage pipe layout should be detailed by Paterson once the structures basement layout has been completed by the architect and structural engineer. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structures. Such exterior structures require additional frost protection, such as 2.1 m of soil cover, or a reduced thickness of soil cover if rigid insulation is used.

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

6.3 Excavation Side Slopes

The side slopes of excavations in the 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 assumed that sufficient room will be available for the greater part of the excavations to be undertaken by open-cut methods (i.e. 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 a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavation side slopes around the building excavation should be protected from erosion by surface water and rainfall events by the use of secured tarpaulins spanning the length of the side slopes or other means along their footprint. Efforts should also be made to maintain dry surfaces at the bottom of excavation footprints and along the bottom of side slopes. Additional measures may be recommended at the time of construction by the geotechnical consultant.

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.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical 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.

Temporary Shoring

The shoring requirements will depend on the depth of the excavation and the proximity of the adjacent structures. However, it should be noted that the observed site conditions may be unsuitable for conventional temporary shoring installation methods. Bouldery conditions are expected to be encountered which can lead to difficulty driving temporary shoring system structures to design termination depths. Furthermore, it may be difficult to develop the required anchor strength in soil conditions due to variations in soil conditions.

If a temporary shoring system is considered, the design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services.

The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s structural design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. However, due to the bouldery glacial till layer, the site may not be suitable for interlocking steel sheet piling. Consideration should be given to a system that can be advanced within boulder soil conditions.

Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability.

The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 6 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System	
Parameter	**Value**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	20
Submerged Unit Weight (γ'), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

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 99% of the material's standard Proctor maximum dry density.

It should generally be possible to re-use the upper portion of the dry to moist (not wet) silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. Any stones greater than 200 mm in their longest dimension should be removed from these materials prior to placement.

The backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce potential differential frost heaving. The backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD.

6.5 Groundwater Control

Based on our observations, it is anticipated that groundwater infiltration into the excavations should be low to moderate and 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.

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 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, 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 Person as stipulated under O.Reg. 63/16.

6.6 Winter Construction

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

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

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

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

Under winter conditions, if snow and ice is present within the blast rock or other imported fill below future basement slabs, then settlement of the fill should be expected and support of a future basement slab and/or temporary supports for slab pours will be negatively impacted and could undergo settlement during spring and summer time conditions. The geotechnical consultant should complete periodic inspections during fill placement to ensure that snow and ice quantities are minimized in settlement-sensitive areas.

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 non-aggressive to slightly aggressive corrosive environment.

6.8 Landscaping Considerations

Tree Planting Restrictions

Paterson completed a soils review of the site to determine applicable tree planting setbacks, in accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines) for trees planted within a public right-of-way (ROW). The review is based on an additional investigation undertaken by Paterson throughout the remainder of the property parcel of 3265 Jockvale Road during January of 2021.

Atterberg limits testing was completed for recovered silty clay samples at selected locations during the additional investigation. Grain size distribution and hydrometer testing was also completed on selected soil samples. The above-noted test results were completed on samples taken at depths between the anticipated underside of footing elevation and a 3.5 m depth below finished grade. The results of our testing are presented in Tables 1 and 2 in Subsection 4.2 and in Appendix 1.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to be less than 40% in all the tested clay samples. In addition, based on the clay content found in the clay samples from the grain size distribution test results, moisture levels and consistency, the silty clay across the subject site is considered low to medium sensitivity clay and cannot be designated as sensitive marine clays.

The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit throughout the subject site where silty clay was encountered (permissible grade raise restriction area) as shown in Drawing PG5636-3 - Tree Planting Setback Plan. It should be noted that footings bearing upon a compact to very dense glacial till deposit will not be subject to tree planting setback restrictions.

Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits may be reduced to 4.5 m for small (mature height up to 7.5 m) and medium size trees (mature tree height 7.5 to 14 m), provided that the condition noted below 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).

It is well documented in the literature, and is our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures. Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e., Manitoba Maples) and, as such, they should not be considered in the landscaping design.

Aboveground Swimming Pools, Hot Tubs, Decks and Additions

The in-situ soils are considered to be acceptable for in-ground swimming pools. Above ground swimming pools must be placed at least 5 m away from the residence foundation and neighboring foundations. Otherwise, pool construction is considered routine, and can be constructed in accordance with the manufacturer`s requirements.

Additional grading around the hot tub should not exceed permissible grade raises. Otherwise, hot tub construction is considered routine, and can be constructed in accordance with the manufacturer’s specifications.

Additional grading around proposed deck or addition should not exceed permissible grade raises. Otherwise, standard construction practices are considered acceptable.

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 structural plan(s) from a geotechnical perspective.
- Review of the geotechnical aspects of the excavation contractor's shoring design, prior to construction, if applicable.
- Review of architectural plans pertaining to foundation and underfloor drainage systems and waterproofing details for elevator shafts.

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:

- Review and inspection of the installation of the foundation drainage systems.
- Observation of all bearing surfaces prior to the placement of concrete.
- 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 by the geotechnical consultant.

All excess soil 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 Minto Communities 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.

LICENSED PROFESSIONAL ENGINEER
June 27, 2023
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Drew Petahtegoose, B.Eng.

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Minto Communities (1 email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ATTEBRERG LIMIT'S TESTING RESILTS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Commercial and Residential Development
3265 Jockvale Road - Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** 2021 January 8

FILE NO. PG5636

HOLE NO. BH 6-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	94.11
Very stiff brown **SILTY CLAY** some organics	AU	1				
	SS	2	83	10	1	93.11
	SS	3	100	7	2	92.11
2.29						
GLACIAL TILL: Compact grey clayey silty sand, with gravel, cobbles and boulders	SS	4	100	5		
	SS	5	44	+50	3	91.11
	SS	6	72	13	4	90.11
	SS	7	13	15	5	89.11
	SS	8	58	16		
6.10					6	88.11
End of Borehole						
(GWL based on site observations at 3.20 m depth - Jan 8, 2021)						

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Commercial and Residential Development
3265 Jockvale Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5636**

REMARKS

HOLE NO. **BH 7-21**

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 8

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
TOPSOIL	0.20	AU	1		
Very stiff brown **SILTY CLAY**		SS	2	75	12
		SS	3	100	10
		SS	4	100	8
		SS	5	100	4
- Grey and stiff by 3.6 m depth	6.55	SS	6	100	7
GLACIAL TILL: Grey silty clay with sand gravel, cobbles and boulders	6.71				
End of Borehole					

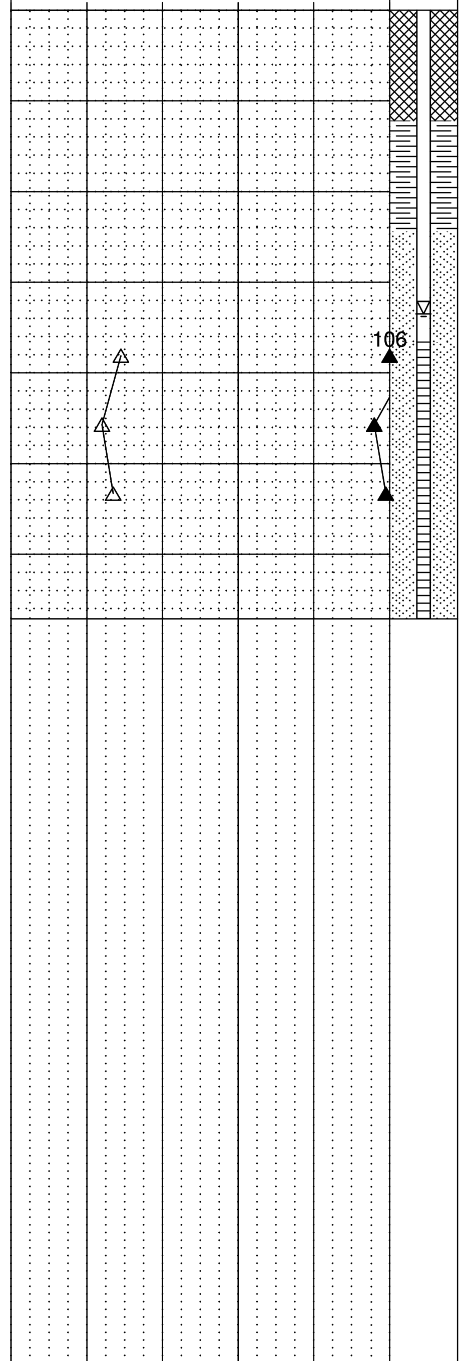
(GWL based on site observations at 3.35 m depth - Jan 8, 2021)

DEPTH (m)	ELEV. (m)
0	93.10
1	92.10
2	91.10
3	90.10
4	89.10
5	88.10
6	87.10

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

Piezometer Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Commercial and Residential Development
3265 Jockvale Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5636**

REMARKS

HOLE NO. **BH 3-21**

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 8

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	102.03
FILL: Brown silty sand some gravel (0.61)		AU	1				
GLACIAL TILL: Brown clayey silty sand some gravel and cobbles		SS	2	71	+50	1	101.03
		SS	3	50	+50		
- Grey by 3.2 m depth						2	100.03
		SS	4	29	41		
(3.40)		SS	5	60	+50	3	99.03

End of Borehole

Practical refusal to augering at 3.40 m depth

(GWL based on site observations at 1.83 m depth - Jan 8, 2021)

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Commercial and Residential Development
3265 Jockvale Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5636

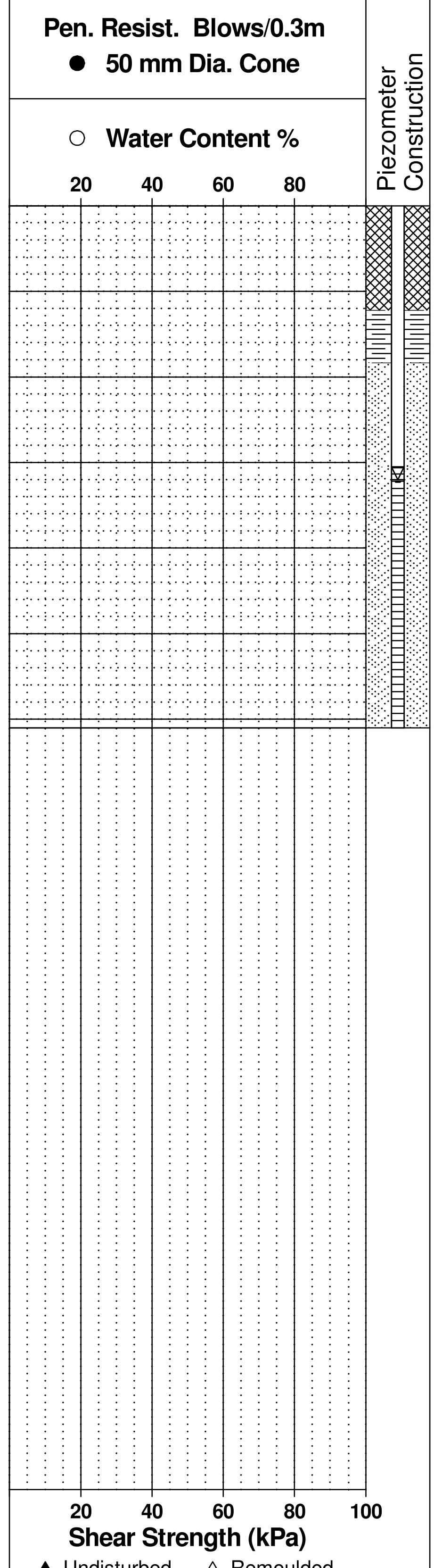
REMARKS

HOLE NO. BH 6-21

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 8

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	94.11
Very stiff brown **SILTY CLAY** some organics	AU	1				
	SS	2	83	10	1	93.11
	SS	3	100	7	2	92.11
2.29						
GLACIAL TILL: Compact grey clayey silty sand, with gravel, cobbles and boulders	SS	4	100	5		
	SS	5	44	+50	3	91.11
	SS	6	72	13	4	90.11
	SS	7	13	15	5	89.11
	SS	8	58	16		
6.10					6	88.11
End of Borehole						
(GWL based on site observations at 3.20 m depth - Jan 8, 2021)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Mixed-Use Commercial and Residential Development
3265 Jockvale Road - Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5636

REMARKS

HOLE NO. BH 7-21

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 8

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	93.10
TOPSOIL	0.20	AU	1				
Very stiff brown **SILTY CLAY**		SS	2	75	12	1	92.10
		SS	3	100	10		
						2	91.10
		SS	4	100	8		
						3	90.10
		SS	5	100	4		
- Grey and stiff by 3.6 m depth						4	89.10
						5	88.10
						6	87.10
	6.55	SS	6	100	7		
GLACIAL TILL: Grey silty clay with sand gravel, cobbles and boulders	6.71						
End of Borehole							

(GWL based on site observations at 3.35 m depth - Jan 8, 2021)

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

○ Water Content %

20 40 60 80

Piezometer Construction

106

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

HOLE NO. BH 1

BORINGS BY CME 55 Power Auger **DATE** 22 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	99.83		
FILL: Brown silty sand with gravel, cobbles, trace clay and boulders (0.30)		AU	1						
FILL: Topsoil with brown silty sand, gravel, trace clay (0.69)		AU	2						
		SS	3	92	78	1	98.83		
		SS	4	100	57	2	97.83		
						3	96.83		
		SS	5	92	45				
						4	95.83		
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	6	0	50+				
						5	94.83		
						6	93.83		
		SS	7	83	27				
						7	92.83		
		SS	8	92	33	8	91.83		
						9	90.83		
(9.75)		SS	9	50	68				
End of Borehole (GWL @ 5.60m-Aug. 30/11)									

Water Content % scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

HOLE NO. BH 2

BORINGS BY CME 55 Power Auger **DATE** 22 August 2011

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.02
TOPSOIL	0.30	AU	1				
Brown **SILTY CLAY** with sand, trace gravel and cobbles	1.65	AU	2				
		SS	3	71	2	1	96.02
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles, boulders, trace clay	2.49	SS	4	71	50+	2	95.02
		SS	5	100	50+		
End of Borehole Practical refusal to augering @ 2.49m depth							

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 22 August 2011

FILE NO. PG2457

HOLE NO. BH 2A

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	97.02
TOPSOIL	0.30							
Brown **SILTY CLAY** with sand, trace gravel and cobbles	1.65						1	96.02
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles, boulders, trace clay - grey by 3.0m depth							2	95.02
							3	94.02
			SS	1	83	50		
							4	93.02
			SS	2	8	26		
							5	92.02
							6	91.02
			SS	3	50	8		
							7	90.02
			SS	4	83	44		
							8	89.02
							9	88.02
	9.75		SS	5	67	39		
End of Borehole (GWL @ 3.95m-Aug. 30/11)								

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

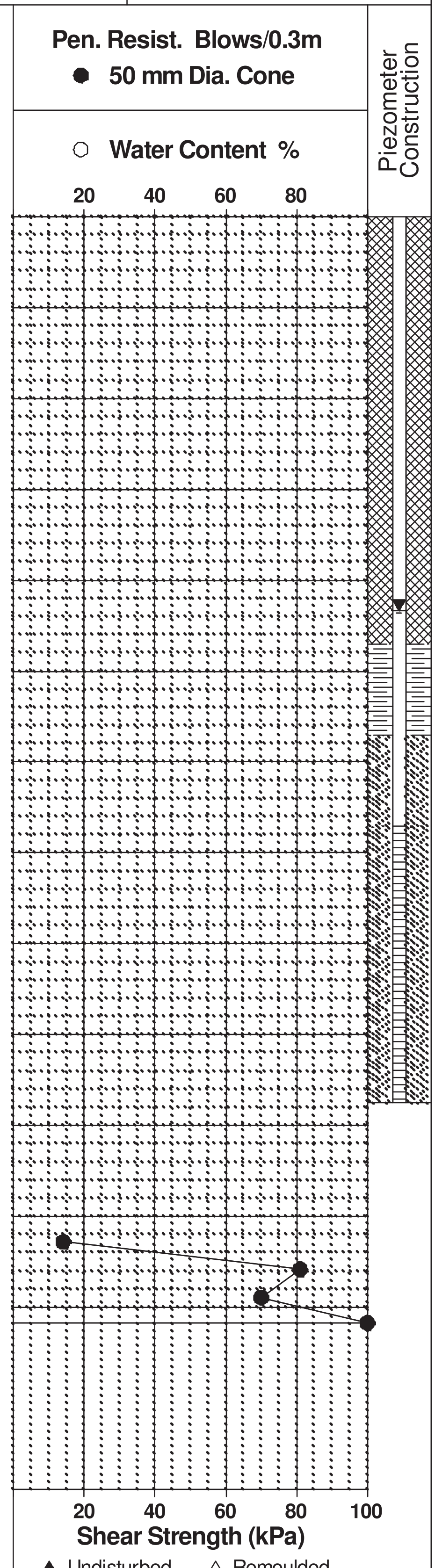
REMARKS

BORINGS BY CME 55 Power Auger **DATE** 22 August 2011

FILE NO. PG2457

HOLE NO. BH 3

SOIL DESCRIPTION	Strata base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	97.23
TOPSOIL	0.25	AU	1				
Brown **SILTY CLAY** with sand, trace gravel	1.22	SS	2	75	10	1	96.23
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	67	27	2	95.23
		SS	4	8	11	3	94.23
- loose to very loose, grey silty sand with clay and gravel, some cobbles and boulders from 3.0 to 5.6m depth		SS	5	100	8	4	93.23
		SS	6	83	1	5	92.23
- dense by 5.6m depth						6	91.23
		SS	7	50	33	7	90.23
		SS	8	75	31	8	89.23
	9.75	SS	9	67	14	9	88.23
Dynamic Cone Penetration Test commenced @ 9.75m depth						10	87.23
Inferred **GLACIAL TILL**	12.19					11	86.23
						12	85.23
End of Borehole							
Practical refusal to DCPT @ 12.19m depth							
(GWL @ 4.34m-Aug. 30/11)							



patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

HOLE NO. BH 4

BORINGS BY CME 55 Power Auger

DATE 22 August 2011

SOIL DESCRIPTION	Sample TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	95.23
FILL: Brown silty clay wtih sand and gravel, trace organics	AU	1				
	AU	2				
0.69						
Very stiff to stiff, brown **SILTY CLAY**	SS	3	100	9	1	94.23
	SS	4	92	8	2	93.23
	SS	5	100	4		
2.97					3	92.23
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles and boulders, trace clay	SS	6	54	21		
	SS	7	21	24	4	91.23
	SS	8	62	19	5	90.23
- grey by 5.3m depth					6	89.23
	SS	9	62	12	7	88.23
	SS	10	21	22	8	87.23
					9	86.23
9.75	SS	11	42	13		
End of Borehole						
(GWL @ 2.94m-Aug. 30/11)						

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

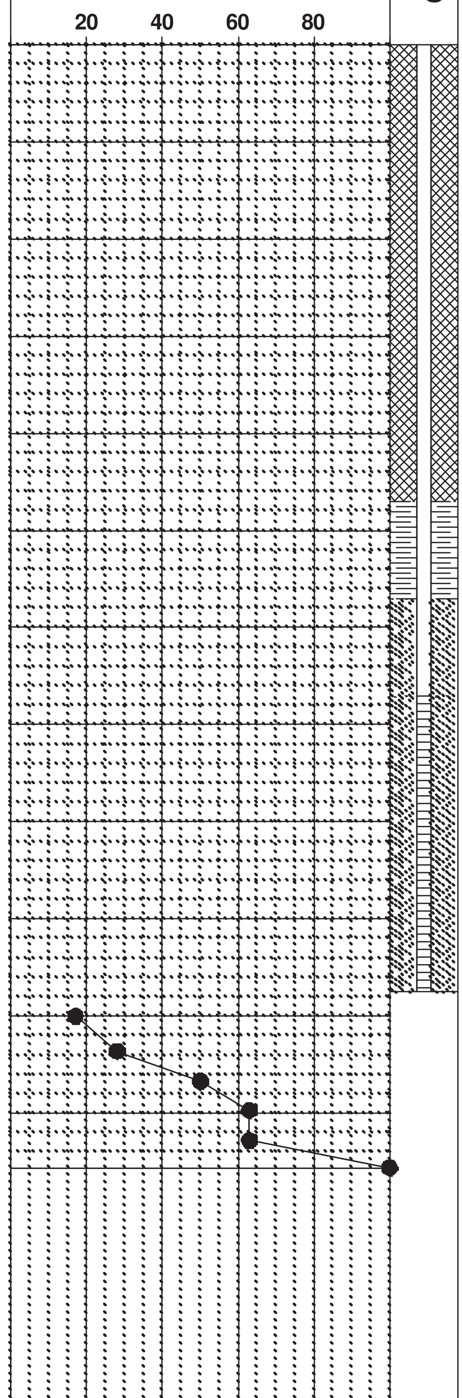
HOLE NO. BH 5

BORINGS BY CME 55 Power Auger **DATE** 23 August 2011

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	101.62
FILL: Brown silty sand with topsoil, trace gravel (to 0.69)	AU	1				
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders, trace clay	SS	2	92	47	1	100.62
	SS	3	100	50+		
					2	99.62
	SS	4	71	50+	3	98.62
					4	97.62
	SS	5	61	32	5	96.62
	SS	6	54	50	6	95.62
					7	94.62
	SS	7	21	37	8	93.62
(to 9.75)	SS	8	92	35	9	92.62
Dynamic Cone Penetration Test commenced @ 9.75m depth. Inferred **GLACIAL TILL** (to 11.56)					10	91.62
					11	90.62
End of Borehole. Practical refusal to DCPT @ 11.56m depth. (Piezometer destroyed - Aug. 30/11)						

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

○ Water Content %



Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

HOLE NO. BH 6

BORINGS BY CME 55 Power Auger **DATE** 22 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	99.95		
FILL: Brown silty sand with gravel, cobbles, trace clay		AU	1						
0.69		AU	2						
						1	98.95		
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	100	51				
		SS	4	92	45	2	97.95		
		SS	5	100	50+				
						3	96.95		
		SS	6	83	45	4	95.95		
						5	94.95		
		SS	7	29	50	6	93.95		
7.09		SS	8	100	50+	7	92.95		

End of Borehole

Practical refusal to augering @ 7.09m depth

(BH dry - Aug. 30/11)

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 23 August 2011

FILE NO. PG2457

HOLE NO. BH 7

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	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	100.32		
FILL: Brown silty sand with gravel, cobbles and boulders (0.30)		AU	1						
		AU	2						
GLACIAL TILL: Dense to compact, brown silty sand with gravel, cobbles and boulders, trace clay		SS	3	45	50+	1	99.32		
		SS	4	100	50+	2	98.32		
		SS	5	100	50+				
		SS	6	67	26	3	97.32		
(3.81) End of Borehole									
Practical refusal to augering @ 3.81m depth									

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

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

HOLE NO. BH 7A

BORINGS BY CME 55 Power Auger

DATE 23 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	100.25
FILL: Brown silty sand with gravel, cobbles and boulders — 0.30						1	99.25
						2	98.25
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, trace clay						3	97.25
						4	96.25
		SS	1	25	64	5	95.25
— 6.48		SS	2	100	50+	6	94.25

End of Borehole

Practical refusal to augering @ 6.48m depth

(GWL @ 4.25m-Aug. 30/11)

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

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development - Chapman Mills Drive
Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Limited.

FILE NO. PG2457

REMARKS

HOLE NO. BH 8

BORINGS BY CME 55 Power Auger **DATE** 23 August 2011

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	96.75
FILL: Brown silty sand with gravel, cobbles, trace clay (to 0.69)		AU	1				
		SS	2	50	76	1	95.75
		SS	3	54	28	2	94.75
		SS	4	54	30	3	93.75
GLACIAL TILL: Very dense to dense, brown silty sand with gravel, cobbles and boulders, trace clay						4	92.75
		SS	5	83	15	5	91.75
		SS	6	50	47	6	90.75
- very loose to loose and grey by 7.2m depth						7	89.75
		SS	7	92	2	8	88.75
						9	87.75
(to 9.75)		SS	8	100	9		
End of Borehole							
(GWL @ 3.12m-Aug. 30/11)							

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 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

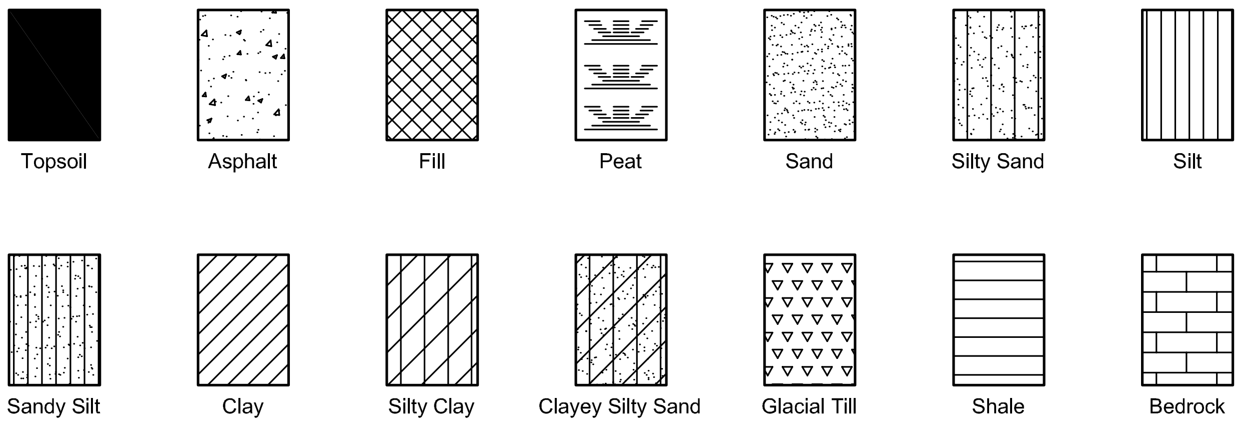
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

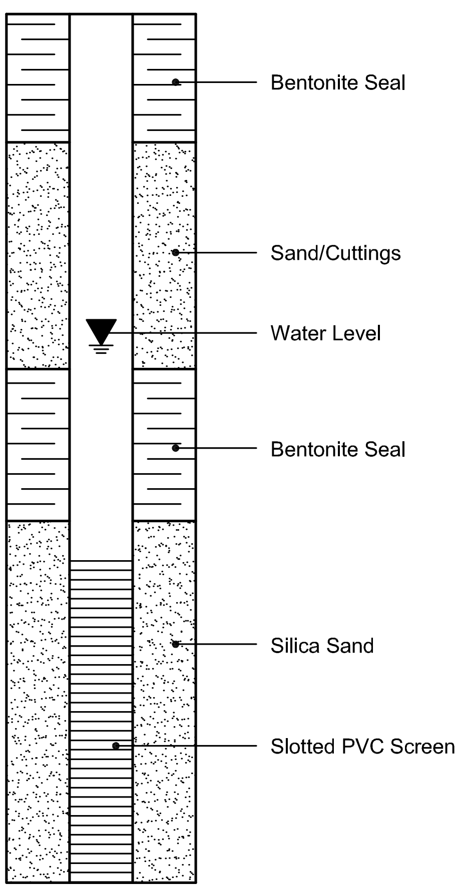
SYMBOLS AND TERMS (continued)

STRATA PLOT

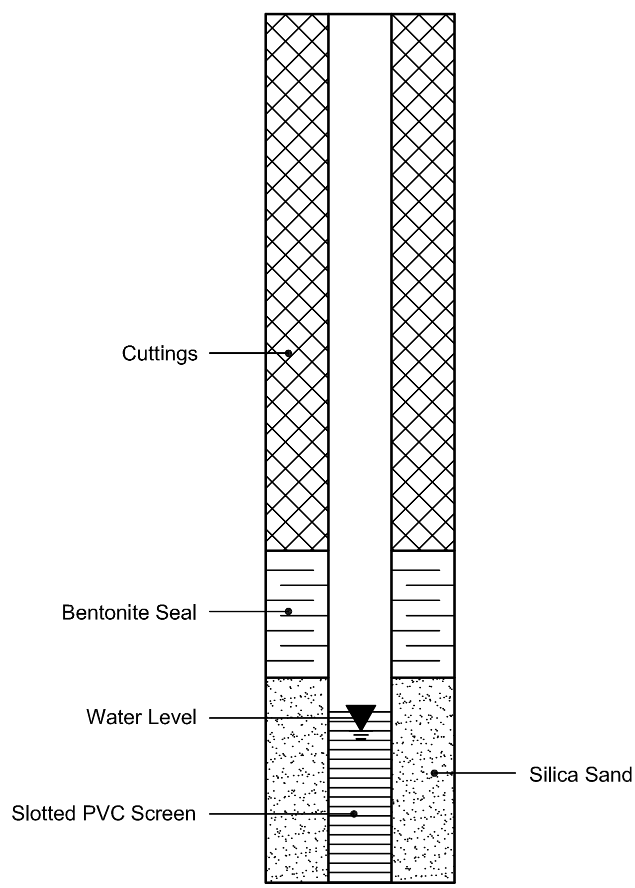


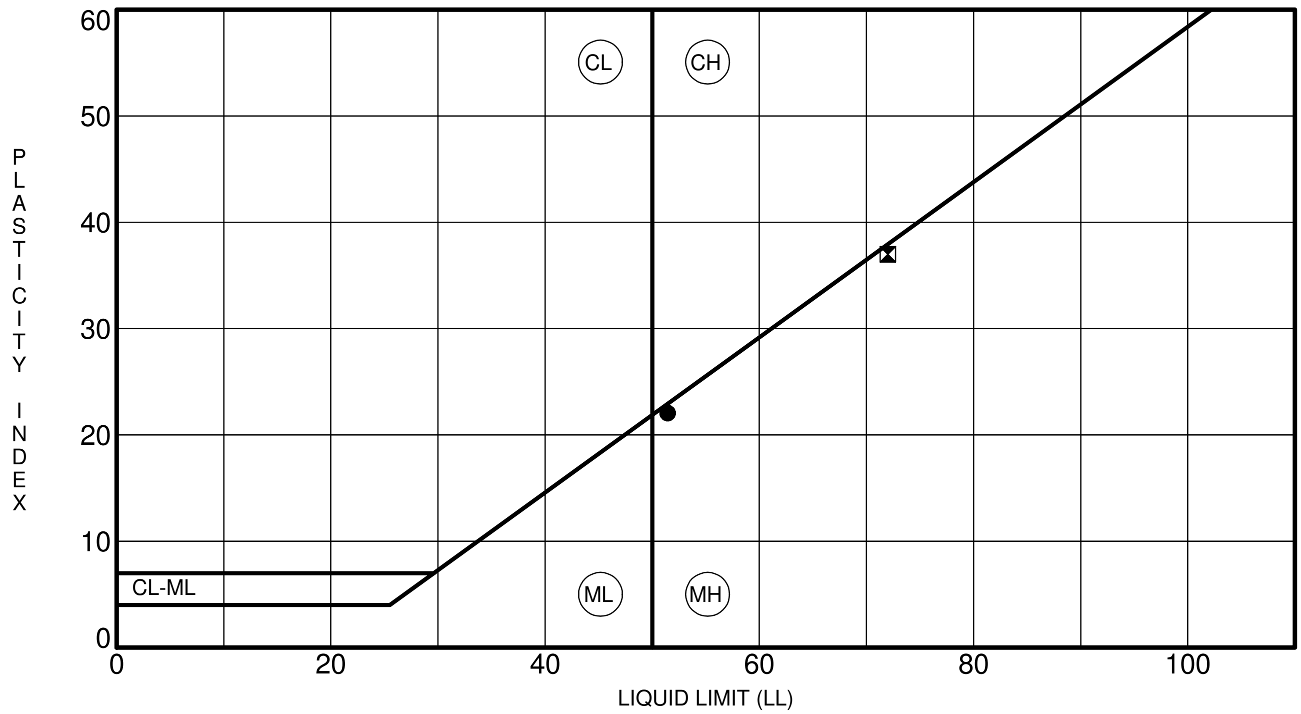
MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION





	Specimen Identification	LL	PL	PI	Fines	Classification
●	**BH 6-21 SS3**	**51**	**29**	**22**	**97.1**	**MH = Inorganic Silt of High Plasticity**
⧗	**BH 7-21 SS4**	**72**	**35**	**37**	**87.3**	**MH = Inorganic Silt of High Plasticity**

CLIENT **Minto Communities Inc.**

FILE NO. **PG5636**

PROJECT **Geotechnical Investigation - Mixed-Use Commercial and Residential Development**

DATE **8 Jan 21**

patersongroup **Consulting Engineers**

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

ATTERBERG LIMITS' RESULTS

Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 11689

Project Description: PG2457

Report Date: 30-Aug-2011

Order Date:23-Aug-2011

	Client ID:	BH6 SS5	-	-	-
	Sample Date:	22-Aug-11	-	-	-
	Sample ID:	1135081-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.2	-	-	-
General Inorganics					
pH	0.1 pH Units	7.9	-	-	-
Resistivity	0.10 Ohm.m	104	-	-	-
Anions					
Chloride	5 ug/g dry	8	-	-	-
Sulphate	5 ug/g dry	9	-	-	-

APPENDIX 2

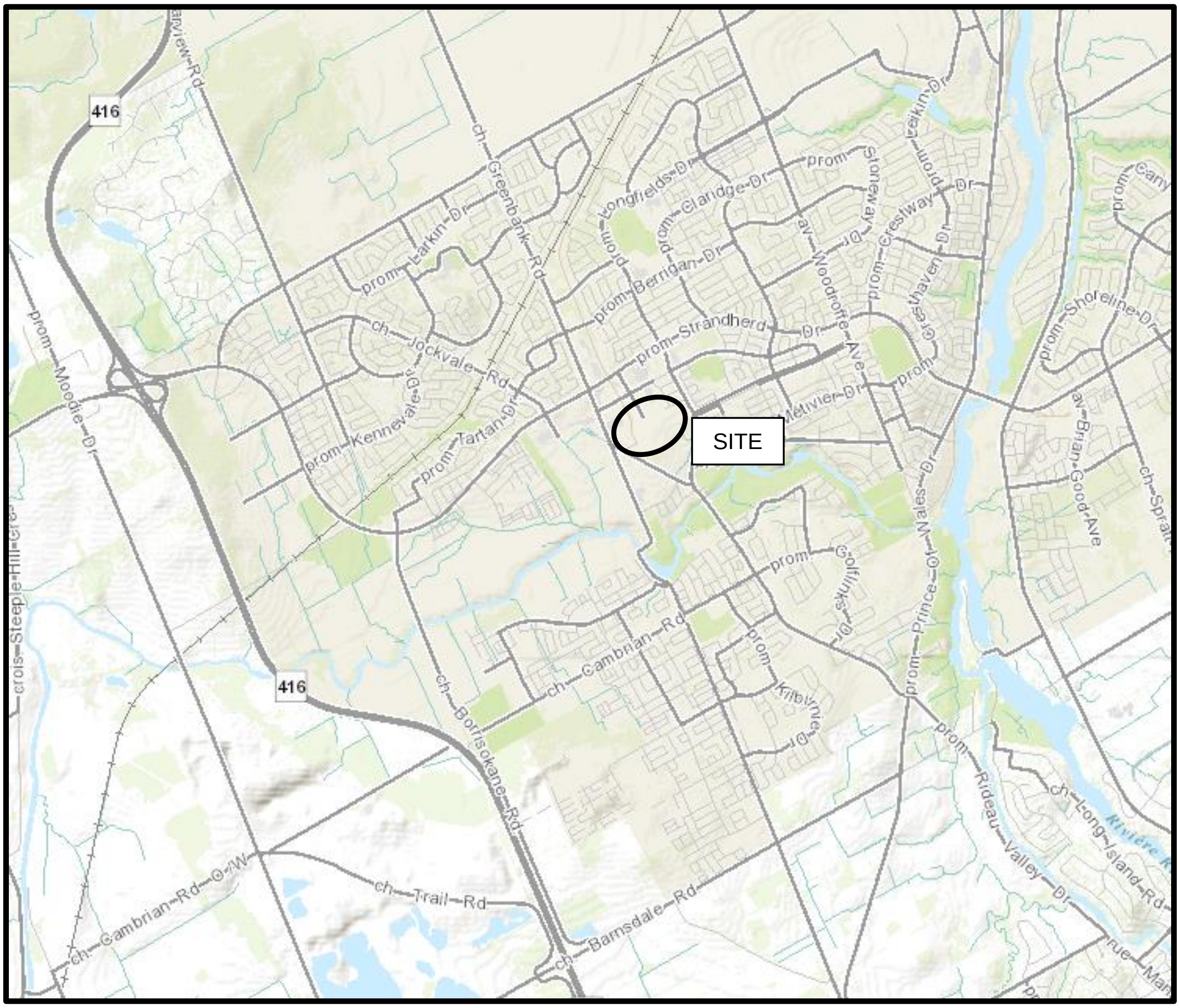


FIGURE 1

KEY PLAN

PATERSON GROUP

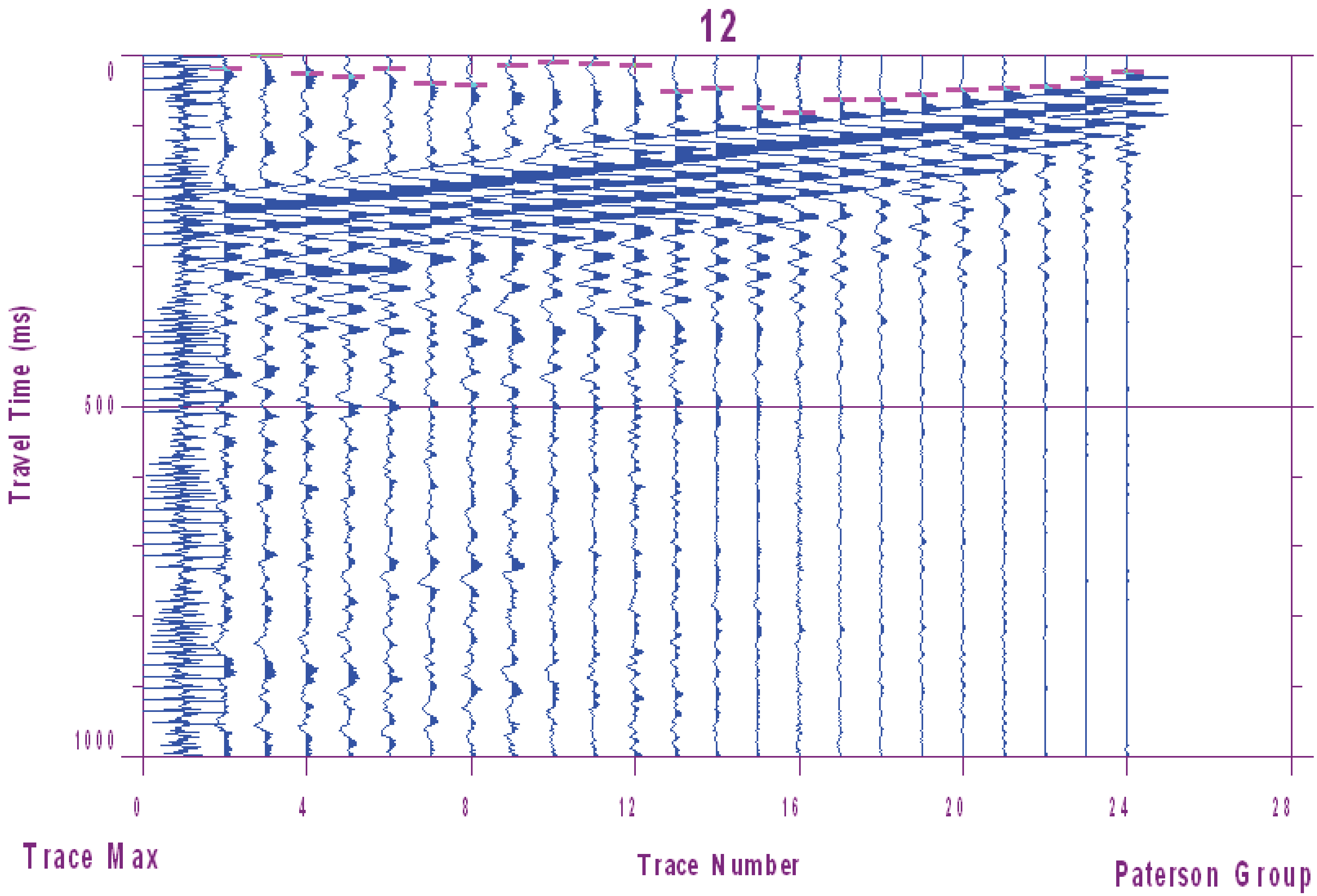


Figure 2 – Shear Wave Velocity Profile at Shot Location 73.5 m

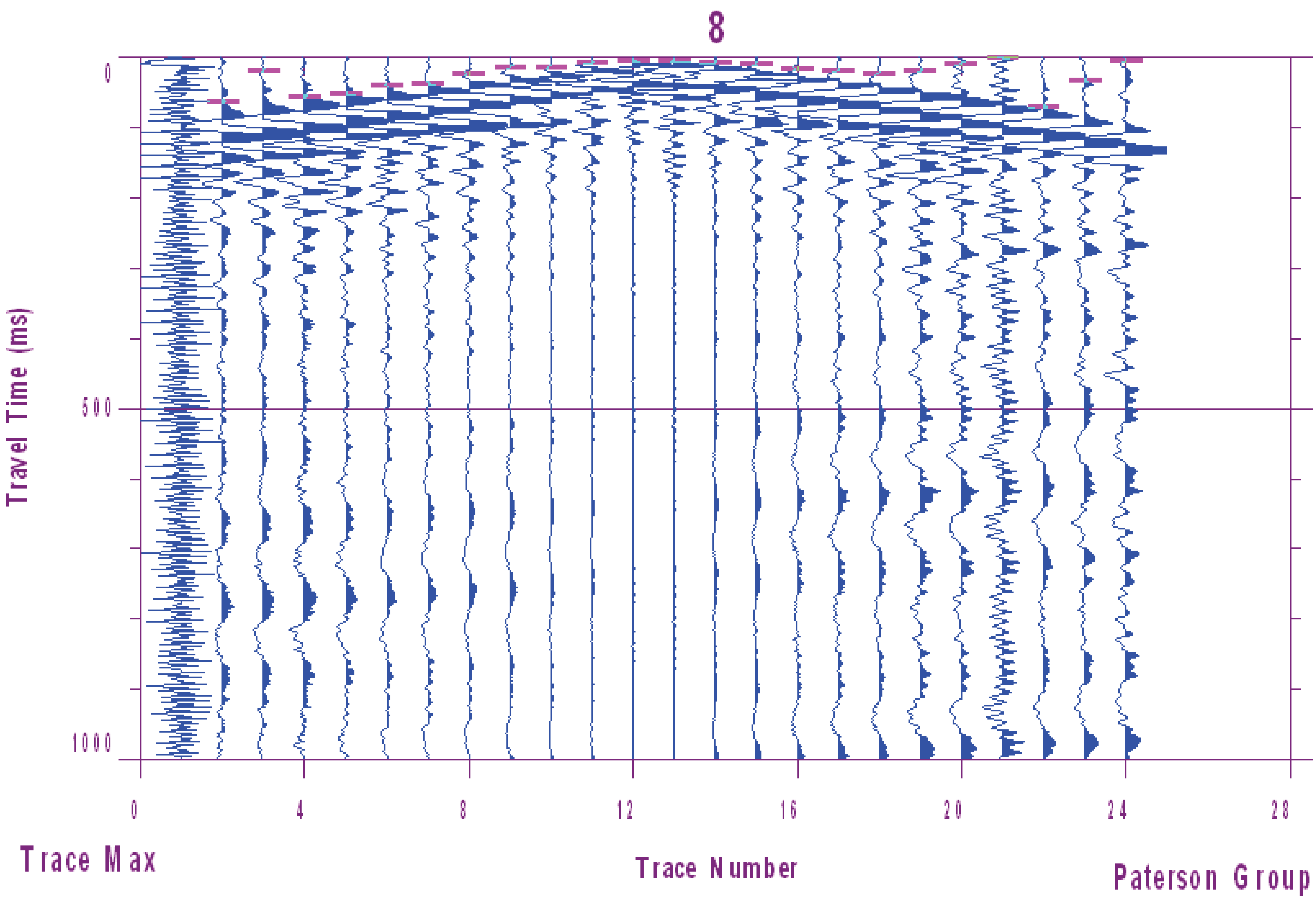


Figure 3 – Shear Wave Velocity Profile at Shot Location 34.5 m

FIGURE 4

Aerial Photograph - 1999

PATERSON GROUP

FIGURE 5

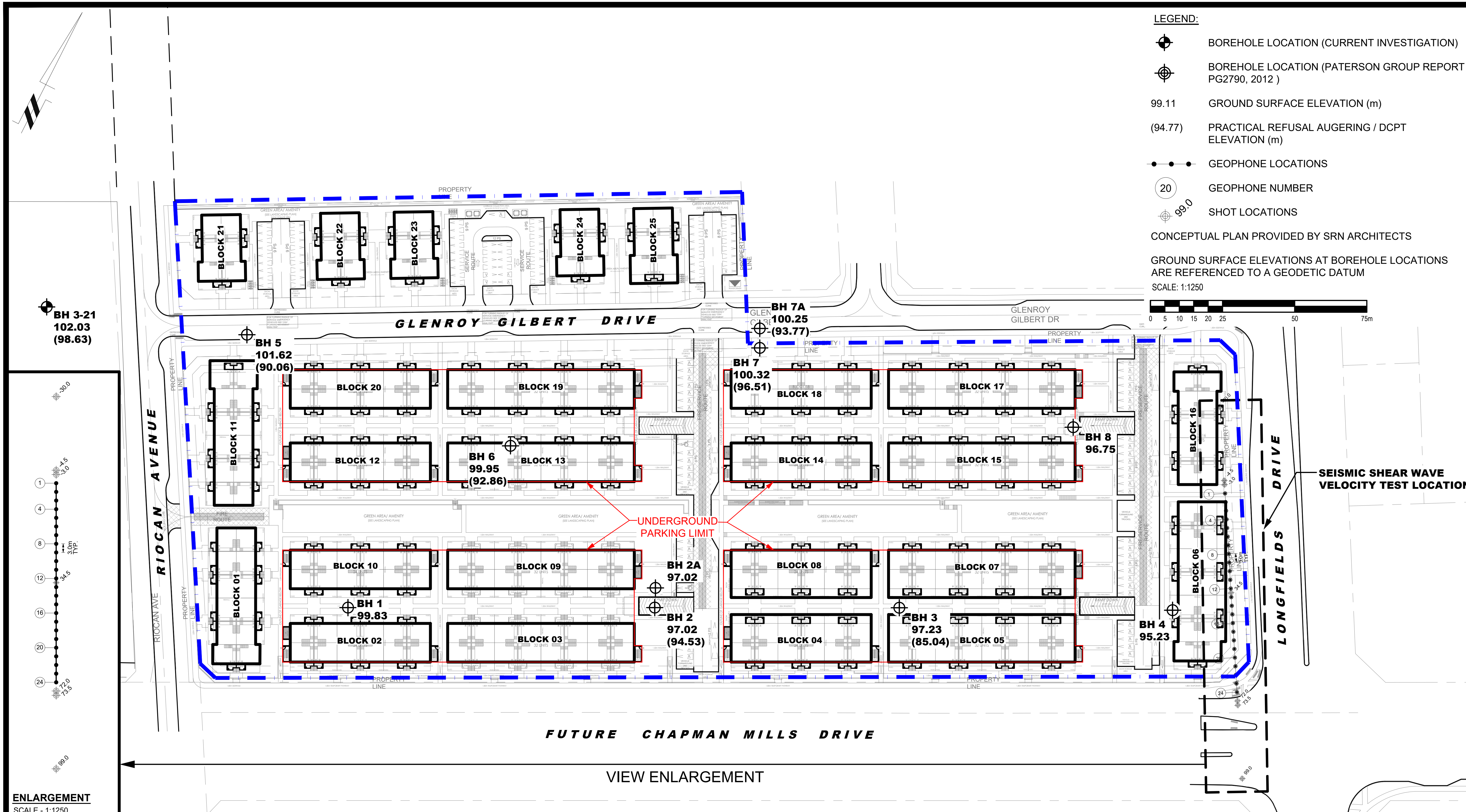
Aerial Photograph - 2011

PATERSON GROUP

FIGURE 6

Aerial Photograph - 2017

PATERSON GROUP



PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	UPDATED DRAWING TO LATEST CONCEPTUAL PLAN	07/06/2023	KB
1	UPDATED LATEST CONCEPTUAL PLAN TO DRAWING	23/11/2021	FC

MINTO COMMUNITIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
CHAPMAN MILLS DRIVE AND RIOCAN AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale:	1:1250	Date:	08/2021
Drawn by:	YA	Report No.:	PG5636-REP.02
Checked by:	DP	Dwg. No.:	PG5636-1
Approved by:	DJG	Revision No.:	2

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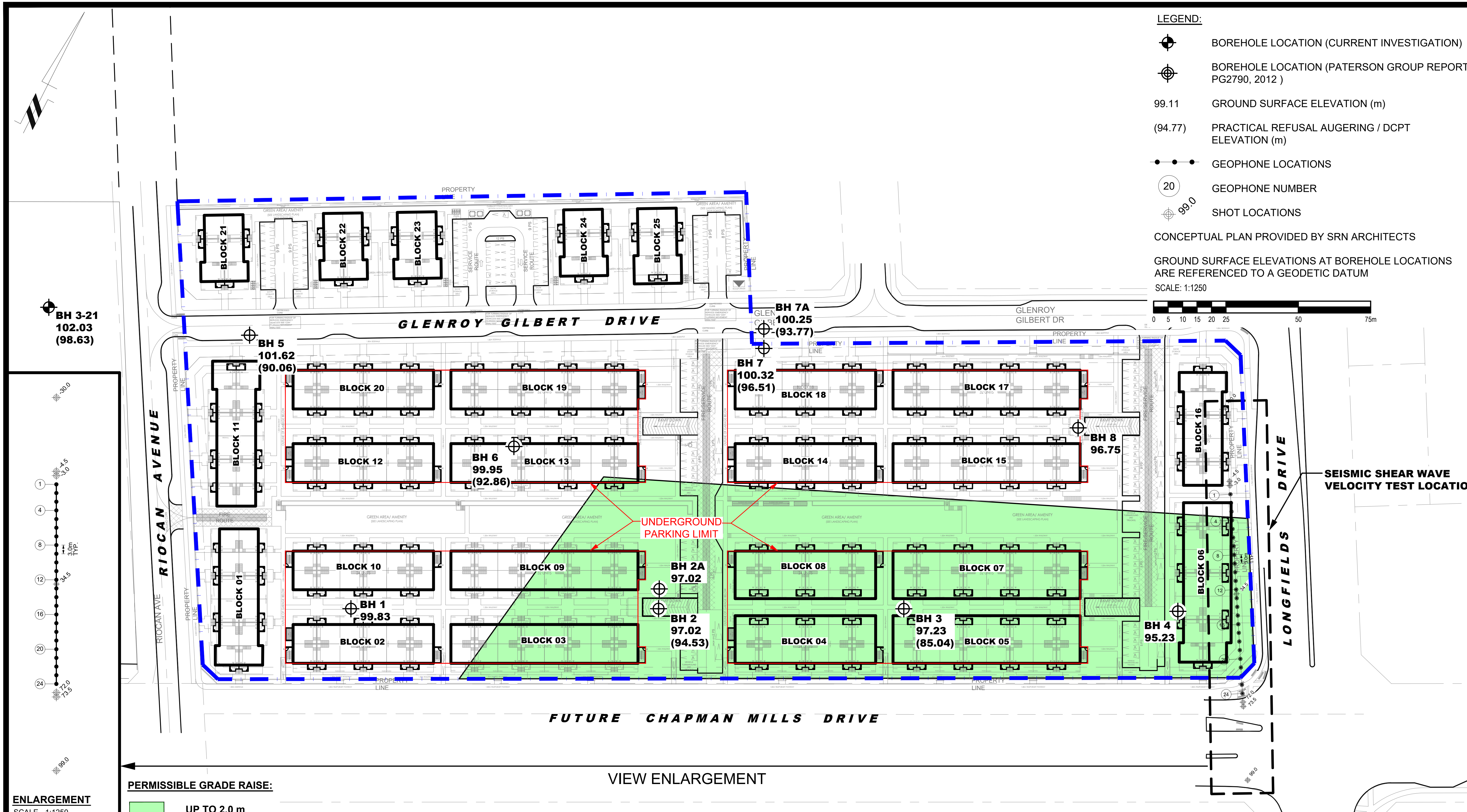

LEGEND:

- BOREHOLE LOCATION (CURRENT INVESTIGATION)
- BOREHOLE LOCATION (PATERSON GROUP REPORT PG2790, 2012)
- 99.11 GROUND SURFACE ELEVATION (m)
- (94.77) PRACTICAL REFUSAL AUGERING / DCPT ELEVATION (m)
- GEOPHONE LOCATIONS
- 20 GEOPHONE NUMBER
- 99.0 SHOT LOCATIONS

CONCEPTUAL PLAN PROVIDED BY SRN ARCHITECTS

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM

SCALE: 1:1250

0 5 10 15 20 25 50 75m

PERMISSIBLE GRADE RAISE:

UP TO 2.0 m

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	UPDATED DRAWING TO LATEST CONCEPTUAL PLAN	07/06/2023	KB
1	UPDATED LATEST CONCEPTUAL PLAN TO DRAWING	23/11/2021	FC

MINTO COMMUNITIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
CHAPMAN MILLS DRIVE AND RIOCAN AVENUE

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale:	1:1250	Date:	08/2021
Drawn by:	YA	Report No.:	PG5636-REP.02
Checked by:	DP	Dwg. No.:	PG5636-2
Approved by:	DJG	Revision No.:	2

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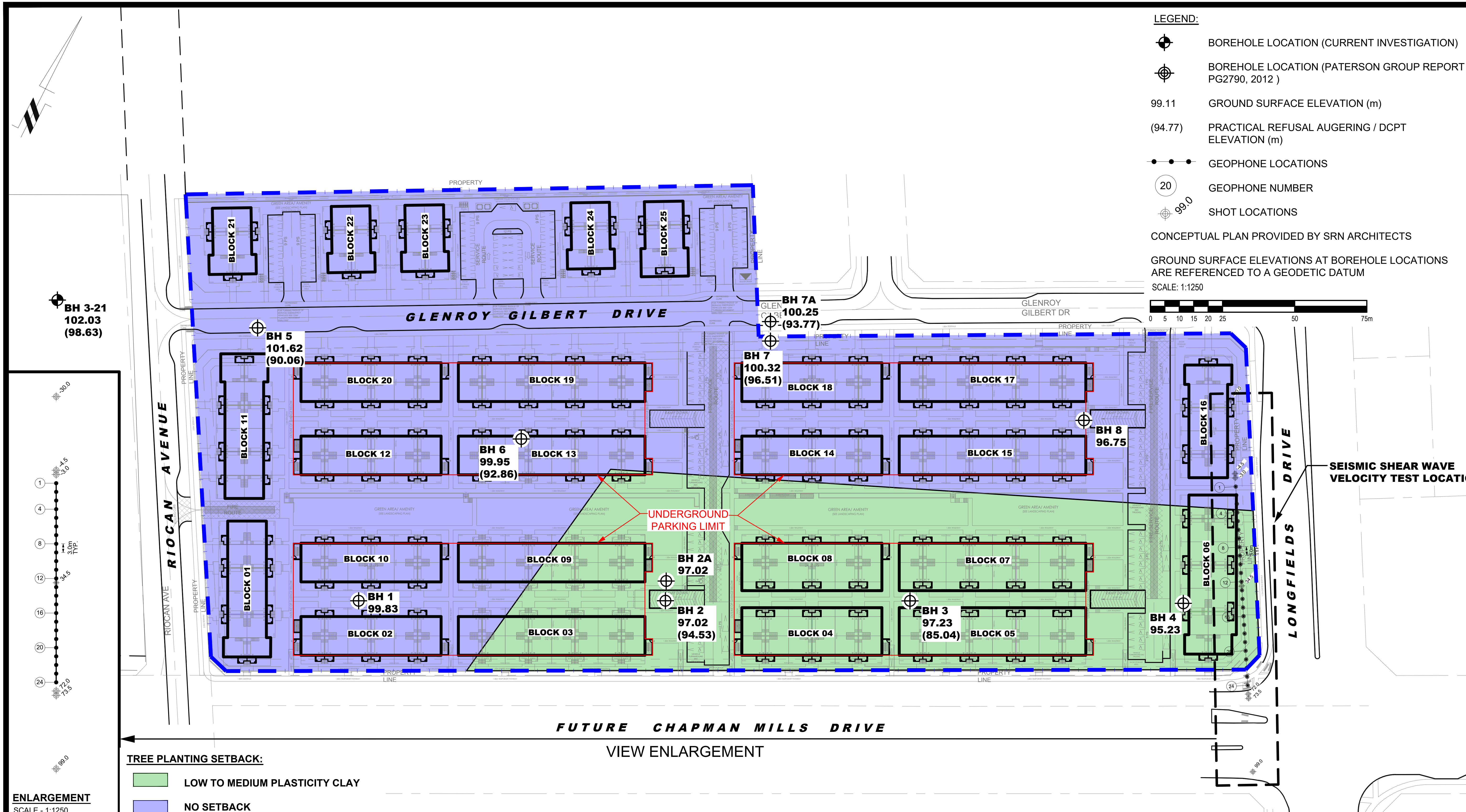

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	UPDATED DRAWING TO LATEST CONCEPTUAL PLAN	07/06/2023	KB
1	UPDATED LATEST CONCEPTUAL PLAN TO DRAWING	23/11/2021	FC

MINTO COMMUNITIES
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
CHAPMAN MILLS DRIVE AND RIOCAN AVENUE

OTTAWA, ONTARIO

Title:

TREE PLANTING SETBACK RECOMMENDATIONS PLAN

Scale: 1:1250	Date: 08/2021
Drawn by: YA	Report No.: PG5636-REP.02
Checked by: DP	Dwg. No.: PG5636-3
Approved by: DJG	Revision No.: 2

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