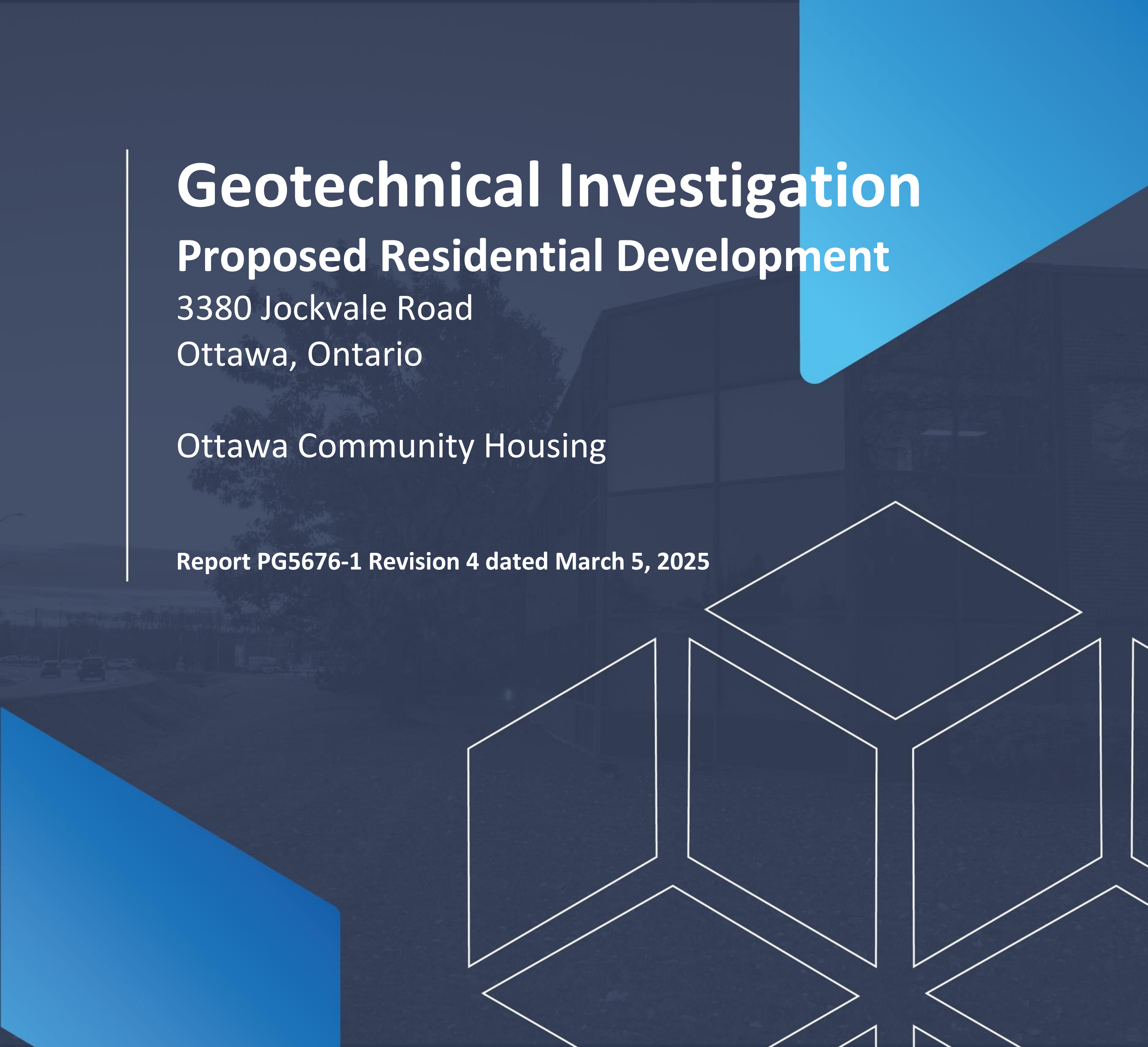
Geotechnical Investigation

Proposed Residential Development

3380 Jockvale Road
Ottawa, Ontario

Ottawa Community Housing

Report PG5676-1 Revision 4 dated March 5, 2025



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1.0 Introduction

Paterson Group (Paterson) was commissioned by Ottawa Community Housing to conduct a geotechnical investigation for the proposed residential development to be located at 3380 Jockvale Road in the City of Ottawa, Ontario (refer to Figure 1 – Key Plan in Appendix 2 of this report).

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 property was not part of the scope of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the latest conceptual drawings, it is understood that the proposed development will consist of two blocks of back-to-back, low-rise, residential buildings of slab-on-grade construction, one back-to-back, low rise, residential building with one basement level, and one mid-rise residential building with one level of underground parking. Associated access lanes, parking areas and landscaped areas are anticipated within the development. It is further anticipated that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the initial geotechnical investigation was conducted carried out by Lopers & Associates on January 6, 2021. At that time, three (3) boreholes were advanced to a maximum depth of 6 m below the existing grade. The borehole locations were distributed in a manner to provide general coverage of the subject site. Two previous geotechnical field investigations were conducted in the vicinity by others on August 17, 2015, and March 14, 2017.

A supplemental geotechnical field program was conducted by Paterson on January 30 and 31, 2025. At that time, a total of three (3) boreholes were advanced to a maximum depth of 9.8 m below the existing ground surface. The purpose of this geotechnical investigation is to assess the overburden and determine the density of the subsurface layers.

A previous supplemental field program was completed by Paterson Group on May 11, 2022. At the time a total of four (4) test pits (TP 1-22 to TP 4-22) were excavated to a maximum depth of 2.3 m.

The test hole locations are presented on Drawing PG5676-1 – Test Hole Location Plan, included in Appendix 2.

All test holes were advanced using a track-mounted auger drill rig and a backhoe excavator for boreholes and test pits, respectively. The fieldwork for the current investigation was conducted by Paterson Group. All fieldwork was conducted under the direction of a Paterson Group senior engineer. The drilling and test pitting procedures consisted of augering and excavating, respectively, to the required depths at the selected locations, sampling, and testing the overburden.

Sampling and In Situ Testing

Soil samples were collected from the test pits by grab samples (G) and 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. All samples were visually inspected and initially classified on site and subsequently placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and grab samples were recovered from the boreholes are shown as AU, SS and G, respectively, on the Soil Profile and Test Data sheets presented 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.

The overburden thickness was evaluated by a dynamic cone penetration test (DCPT) at the location of boreholes BH 2A-25.

The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at the tip, using a 63.5 kg hammer falling from a height of 760 mm. The number of blows required to drive the cone into the soil is recorded for each 300 mm increment.

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

Groundwater

Boreholes BH 1-25, BH 2A-25, and BH 3-25 were fitted with a flexible polyethylene standpipe to allow groundwater level monitoring. The groundwater level readings were obtained after a suitable stabilization period subsequent to the completion of the field investigation.

Groundwater infiltration was measured within the side walls of the test pits at the time of excavation during the previous supplemental field program. Furthermore, field observations were noted throughout the field programs, and moisture levels in the soil samples were subjected to laboratory review.

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 ground surface elevation at each test hole location were surveyed by Paterson using a high-precision 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 PG5676-1 – 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 one (1) shrinkage test, one (1) grain size distribution, and hydrometer test, two (2) Atterberg limits tests, and moisture content testing were completed on selected soil samples. The laboratory testing results are further discussed in Subsection 4.2 and presented in Appendix 1.

3.4 Analytical Testing

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

4.0 Observations

4.1 Surface Conditions

Fill has been placed across the majority of the subject site, with the remainder being grass covered. Fill piles are present in the western half of the property. The subject site is bordered by Jockvale Road to the north, a low-rise residential area property and Bending Way to the west, undeveloped land, the Jock River to the south, and Longfields Drive to the east. The existing ground surface is relatively flat across the site with an approximate geodetic elevation of 91.3 to 92.5 m.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile at the test hole locations consists of fill, or topsoil underlain by fill consisting of brown silty sand to silty clay with some sand and gravel. The fill was generally observed to be underlain by compact to very dense glacial till, or by compact to very dense silty sand or very stiff, brown silty clay followed by the glacial till deposit.

It should be noted that the glacial till was noted to be found in a loose state at geodetic elevations between 88.5 and 89.5 m within borehole BH 2-21. However, the remainder of the boreholes indicate a compact to very dense glacial till deposit.

Furthermore, BH 15-2 from a previous investigation by others noted the presence of a sand and gravel deposit at geodetic elevations between 85.5 and 87.8 m, which was not encountered in nearby boreholes conducted by Paterson. Given the description of the sand and gravel deposit presented in the 15-2 borehole log, it is believed that the deposit is misidentified and actually consists of compact to very dense glacial till.

Practical refusal to DCPT testing was encountered at a depth of 10.1 m below the existing ground surface at borehole BH 2A-25 location at the subject site.

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

Bedrock

Bedrock was cored in borehole BH 15-2 from a depth of 11.6 m to a depth of 14.5 m and consisted of thin to thickly bedded, grey dolomite bedrock.

Based on available geological mapping, bedrock depth ranges from 5 to 15 m below existing ground surface. The site is located on the border of the March formation and Gull River formation, which consists of interbedded dolomite and sandstone, and interbedded limestone and dolomite, respectively.

Atterberg Limit and Shrinkage Tests

Atterberg limit testing, as well as associated moisture content testing was completed on the recovered silty clay samples at selected locations throughout the subject site. The results of the Atterberg limits are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 – Atterberg Limits Results

Test Hole Number	Depth (m)	LL (%)	PL (%)	PI (%)	Classification
TP 3-22	2.0-2.3	48	28	20	CH
TP 4-22	2.0-2.3	44	24	20	CH

Note: LL : Liquid Limit; PL: Plastic Limit; PI: Plasticity Index ;
CH : Inorganic Clay of High Plasticity; MH: Inorganic Silt of Plasticity

The results of the shrinkage limit test indicate a shrinkage limit of 19.5% and a shrinkage ratio of 1.78.

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was also completed on one (1) selected soil sample. The result of the grain size analysis is summarized in Table 2 on the following page and presented in the Grain-size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 – Summary of Grain Size Distribution Analysis

Test Hole Number	Sample	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
TP 3-22	G1	2.5	20.5	40.5	36.5

4.3 Groundwater

The groundwater level (GWL) readings are presented in Table 3. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled boreholes.

Groundwater conditions can also be estimated based on the observed colour, moisture levels, and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater level can be expected at an approximate geodetic elevation of **87.0 to 88.0 m**. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore levels could differ at the time of construction.

Table 3 – Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 1-25	91.65	4.41	87.24	February 7, 2025
BH 2A-25	91.77	Blocked	N/A	
BH 3-25	91.76	4.23	87.53	
BH 15-2	91.59	4.10	87.49	August 24, 2015
BH 15-3A	91.34	2.98	88.36	
TP 1-22	92.50	Dry	-	May 11, 2022
TP 2-22	91.91	Dry	-	
TP 3-22	92.41	Dry	-	
TP 4-22	91.50	Dry	-	

Note: Ground surface elevations at borehole locations were surveyed by Paterson and are referenced to a geodetic datum.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. It is expected that the proposed buildings will generally be founded on conventional shallow footings over an undisturbed, very stiff, brown silty clay or a compact to dense glacial till bearing surface.

Due to the presence of a silty clay layer encountered at test holes TP 2-22 and BH 17-5, a portion of the subject site will be subjected to a grade raise restriction. The permissible grade raise restrictions are further discussed on Subsection 5.3 and presented in Drawing PG5676-2 – Permissible Grade Raise Plan in Appendix 2.

Where the silty sand subgrade below buildings and paved areas is found to be in a loose state of compaction, proof-rolling using a suitably sized roller is required to be completed under dry conditions and above freezing temperatures to achieve adequate compaction levels and under the full supervision and approval of Paterson at the time of construction.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil, and fill, containing significant amounts of deleterious or organic materials, should be stripped from under any building, paved areas, pipe bedding and other settlement sensitive structures.

It is anticipated that existing fill within the proposed building footprint, free of deleterious material and significant amounts of organics, and approved by the geotechnical consultant at the time of construction, can be left in place below the proposed building footprints outside of lateral support zones for the footings.

It is recommended that the existing fill layer be proof-rolled by a vibratory roller making several passes under dry and above freezing conditions and approved by the geotechnical consultant at the time of construction. Any poor performing areas noted during the proof-rolling operation should be removed and replaced with an approved fill.

Fill Placement

Fill used for grading beneath the proposed buildings should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II.

This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building and paved areas should be compacted to at least 98% of the material's standard Proctor maximum dry density (SPMDD).

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

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

5.3 Foundation Design

Bearing Resistance Values

Pad footings, up to 5 m wide, and strip footings up to 3 m wide, placed on an undisturbed, silty clay bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **150 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

Footings placed on an undisturbed, very dense silty sand or glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **350 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

An undisturbed soil bearing surface consists of a surface from which 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.

Footings designed using the above noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlement of 25 and 20 mm, respectively.

Raft Foundation

For support of the proposed multi-storey building, consideration can be given to using a raft foundation due to the expected building loads.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal associated with one underground parking level.

The bearing resistance value at SLS (contact pressure) of **275 kPa** is considered acceptable for a raft supported on the undisturbed, very dense glacial till. The factored bearing resistance (contact pressure) at ULS can be taken as **410 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

Base on a single underground parking level or more it is expected that the raft foundation will be installed on the glacial till deposit. The modulus of subgrade reaction was calculated to be **11 Mpa/m** for a contact pressure of **275 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

The proposed building can be designed using the above parameters with a total and differential settlement of 25 and 15 mm, respectively.

Deep Foundation (End-Bearing Piles)

If the above-noted bearing resistance values for footings or a raft foundation are not sufficient for the mid-rise building design, a deep foundation system consisting of end-bearing piles, hydraulically driven to refusal in the bedrock surface, will be required. For deep foundations, concrete-filled steel pipe piles are generally used in the Ottawa area. Applicable pile resistance values at ultimate limit states are given in Table 4 on the following page. A resistance factor of 0.4 has been incorporated into the factored ULS values.

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

Table 4 – Pile Foundation Design Data

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

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

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

Should grade raises occur at the site as part of the proposed development, downdrag loads should be considered on the piles. Based on the available subsurface information, it is expected that the deep foundations will be installed through approximately 8 to 12 m of glacial till.

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 capacity criterion and does not affect the geotechnical capacity of the piles or caissons. 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 raft foundation, the structural strength of the embedded pile or caisson 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 or caisson structure is well confined.

The 4th edition of the Canadian Foundation Engineering Manual recommends that the allowable structural axial capacity of piles or caissons 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 or caisson material strength (steel yield and concrete 28-day compressive strength).

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 or glacial till 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 soil.

Settlement and Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction is recommended for footings and local roadways and/or parking areas founded over a silty clay subgrade. A permissible grade raise restriction of **2 m** is recommended for housing, which can be increased by 0.5 m for local roadways and/or parking areas.

The approximate area which is anticipated to require a permissible grade raise recommendation is presented in Drawing PG5676-2 – Permissible Grade Raise Plan in Appendix 2.

A post-development groundwater lowering of 0.5 m was considered in the permissible grade raise calculations. 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.

Proof Rolling and Subgrade Improvement Below Footings

Where the silty sand glacial till bearing surface for foundations is found to be in a loose state, as determined by Paterson at the time of construction, a proof rolling program will be required for the bearing surface using suitable compaction equipment prior to forming for foundations. Improving the bearing surface compaction will provide a suitable bearing medium. The proof rolling program should be completed under dry conditions and above freezing temperatures and reviewed and approved by Paterson.

Depending on the looseness and degree of saturation at the time of construction, other measures (additional compaction, dewatering, mud-slab, sub-excavation and reinstatement of crushed stone fill) may be recommended to accommodate site conditions at the time of construction. These considerations should be evaluated at the time of construction by Paterson on a footing-specific basis.

5.4 Design for Earthquakes

Seismic shear wave velocity testing was completed within the subject site to accurately determine the applicable seismic site classification for the proposed buildings in accordance with Ontario Building Code 2024 (OBC 2024). The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided on Figures 2 and 3 in Appendix 2 of the present report.

Field Program

The seismic array was located within the proposed mid-rise building footprint at the subject site and as presented in Drawing PG5676-1 – Test Hole Location Plan attached to the present report. Paterson field personnel placed 18 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were connected by a geophone spread cable to a Geode 18 Channel seismograph.

The seismograph was also connected to a laptop computer 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 were 15, 10 and 2 m away from the first and last geophone.

Data Processing and Interpretation

Interpretation of 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, reflected 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 proposed foundation of the buildings. 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 **400 m/s**, while the bedrock shear wave velocity is **2,188 m/s**. Further, the testing results indicated the average overburden thickness to be approximately 10 m.

Site Class for Buildings Founded on Slab on Grade Construction

For slab on grade construction, it is expected that conventional shallow footings will be founded on soil and more than 3 m of softer materials are expected between the bedrock surface and the underside of footing elevation. For the above-noted design, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in OBC 2024 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{10\ m}{400\ m/s} + \frac{20\ m}{2{,}188\ m/s}\right)}$$

$$V_{s30=}\ 878\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} for the proposed buildings with slab-on-grade construction is **878 m/s**. Based on this, a **Site Designation X_{760}** is applicable for the design of buildings with slab on grade construction as per OBC 2024.

Site Class for Buildings with One Underground Level

Buildings designed with one underground level with foundations consisting of conventional footings, raft slab, or end-bearing piles, where the underside of footing, raft slab or top of pile will be founded on soil and bedrock is anticipated to be located more than 3 m below the founding depth, the V_{s30} was calculated using the standard equation for average shear wave velocity provided in OBC 2024 and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{7\ m}{400\ m/s} + \frac{23\ m}{2{,}188\ m/s}\right)}$$

$$V_{s30=}\ 1{,}071\ \ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity V_{s30} for the proposed buildings with one underground level is **1,071 m/s**. Based on this, as per OBC 2024, a **Site Designation X_{760}** is applicable for the design of buildings founded on conventional footings, raft slab, or end-bearing piles, where approximately 7 m of softer materials are expected between the underside of footing, raft slab or top of pile and the bedrock surface.

The soils underlying the subject site are not susceptible to liquefaction.

5.5 Slab-on-Grade Construction

With the removal of all topsoil and deleterious material, containing organic matter, within the footprints of the proposed buildings, the undisturbed, very stiff, brown silty clay or compact to dense glacial till subgrade, approved by the geotechnical consultant at the time of excavation, will be considered an acceptable subgrade surface on which to commence backfilling for floor and basement slab construction.

Where the subgrade consists of existing fill, a vibratory drum roller should complete several passes over the subgrade surface as a proof-rolling program. Any poor performing areas should be removed and reinstated with an engineered fill, such as Granular B Type II.

It is recommended that the upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade and 19 mm clear crushed stone for basement slab. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 18 kN/m^3.

However, undrained conditions are anticipated (i.e. below the groundwater level). Therefore, the applicable effective (undrained) unit weight of the retained soil can be taken as 11 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

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 $K_o \cdot q$ 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 = $(1.45\text{-}a_{max}/g)a_{max}$
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 0.32g according to the 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 the OBC 2012.

5.7 Pavement Design

Car only parking areas, local roadways and access lanes are anticipated at the subject site. The proposed pavement structures are presented in Tables 5 and 6 below and on the following page.

Table 5 – Recommended Pavement Structure – Car Only Parking Areas

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

Table 6 – Recommended Pavement Structure – Local Roadways and Access Lanes

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 existing 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.

For design purposes, it is recommended that the rigid pavement structure for the lower level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 7 below.

Table 7 – Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**32 MPa Concrete**
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE – Either existing fill, in-situ soil, or OPSS Granular B Type I or II material placed over in-situ soil or fill.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level.

The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example: a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% 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 silty clay is anticipated at subgrade level, consideration should be given to installing subdrains during the pavement construction. The subdrain inverts should be approximately 300 mm below subgrade level and run longitudinal along the curb lines. The subgrade surface should be crowned to promote water flow to the drainage lines.

6.0 Design and Construction Precautions

6.1 Foundation Waterproofing, Drainage and Backfill

Paterson has provided the following preliminary recommendations for foundation drainage and backfill. Comprehensive foundation drainage and backfill recommendations should be completed for the mid-rise residential building once structural, mechanical and civil engineering drawings are made available.

Slab on Grade and Low-Rise Structures

Perimeter Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed buildings. The system should consist of a 150 mm diameter, geotextile-wrapped, perforated, and corrugated plastic pipe, surrounded on all sides by 150 mm of 10 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structures. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Sub-Slab Drainage

For buildings with a below-grade level, sub-slab drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm perforated pipes be placed at approximate 6 m centres. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Mid-Rise Structure(s)

Foundation Waterproofing – Groundwater Suppression System

It is recommended that a groundwater suppression system be provided for the proposed mid-rise building. It is expected that a portion of the building shored while the remaining sides of the building will be excavated with an open excavation methodology. Therefore, it is recommended that the groundwater suppression system consist of the following:

- ☐ For the shoring side, a waterproofing membrane should be placed against the shoring system between underside of footings and an elevation of 87.5 m below existing ground surface. For the remaining portions of the building, the waterproofing should be installed against the soil side.

- ☐ The height of the waterproofing layer should be confirmed once the excavation is completed and the water infiltration is confirmed. This will allow construction to proceed without imposing groundwater lowering within the surrounding area of the proposed buildings in the short and long term conditions.

- ☐ A composite drainage membrane (DeltaDrain 6000, MiraDrain G100N or equivalent) should be placed against the HDPE face of the waterproofing membrane with the geotextile layer facing the waterproofing layer from finished ground surface to the top of the footing.

- ☐ The foundation drainage boards should be overlapped such that the bottom end of a higher board is placed in front of the top end of a lower board. All endlaps of the drainage board sheets should overlap abutting sheets by a minimum of 150 mm. All overlaps should be sealed with a suitable adhesive and/or sealant material approved by the geotechnical consultant. It is highly recommended that the drainage board rolls be installed horizontally rather than vertically to minimize the number of vertical joints forming between the rolls.

- ☐ It is recommended that 150 mm diameter PVC sleeves at 6 m centers be cast in the foundation wall at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The sleeves should be connected to openings in the HDPE face of the drainage board layer. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area via an underfloor and interior drainage pipe system.

- ☐ The top endlap of the foundation drainage board should be provided with a suitable termination bar against the foundation wall to mitigate the potential for water to perch between the drainage board and foundation wall.

Perimeter Drainage

The perimeter and underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and redirect water from the building's foundation drainage system to the buildings sump pit(s). The perimeter and underfloor drainage pipe should consist of a 150 mm diameter corrugated perforated plastic pipe sleeved with a geosock.

Sub-Slab Drainage

The underfloor drainage pipe should be placed in each direction of the basement floor span and connected to the perimeter drainage pipe.

The interior drainage pipe should be provided with tee-connections to extend pipes between the perimeter drainage line and the HDPE-face of the composite foundation drainage board via the foundation wall sleeves. The spacing of the underfloor drainage system should be confirmed by Paterson once the foundation layout and sump system location has been finalized.

Foundation Raft Slab Construction Joints

Where a raft is utilized, it is expected that the raft slab will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the raft slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Elevator Pit Waterproofing

The elevator shaft exterior foundation walls should be waterproofed to avoid any infiltration into the elevator pit. It is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) be applied to the exterior of the elavator shaft foundation wall.

The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the raft slab and down to the top of the footing in accordance with the manufacturer's specifications. A continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the interface between the concrete base slab below the elevator shaft foundation walls.

The 150 mm diameter perforated corrugated pipe underfloor drainage should be placed along the perimeter of the exterior sidewalls and provided a gravity connection to the sump pump basin or the elevator sump pit.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free-draining, non-frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a composite drainage system, such as Delta Drain 6000 or Miradrain G100N. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Finalized Drainage and Waterproofing Design

Paterson should be provided with the finalized structural and architectural drawings for each building to provide a building specific waterproofing and drainage design which includes the above noted recommendations. The design will provide recommendations for other items such as minimum pipe spacings, pipe mechanical connections below grade, transitioning from blind to double sided pours (if applicable), etc.

6.2 Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. Generally, a minimum of 1.5 m thick soil cover (or an equivalent combination of soil cover and foundation insulation) should be provided in this regard.

Exterior unheated footings (such as canopy footings or underground parking ramps) as well as interior footings that are exposed to exterior temperatures for prolonged periods of time (such as entrances to underground parking), are more prone to deleterious movement associated with frost action than the exterior walls of the structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation. Paterson can provide detailed insulation design recommendations once architectural, structural and civil design drawings are available.

6.3 Excavation Side Slopes

The side slopes of excavations in the overburden material 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 undertaken 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 excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. 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 shoring system could consist of a soldier pile and lagging system or steel sheet piles. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. This system could be cantilevered, anchored or braced.

The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of extending the piles into the bedrock through pre-augered holes, if a soldier pile and lagging system is the preferred method.

The earth pressures acting on the temporary shoring system may be calculated with the parameters presented in Table 8 below.

Table 8 – Soils Parameter for Shoring System Design

Parameters	Values
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 is 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.

Paterson can provide a shoring design under a separate cover, once the finalized architectural drawings and an excavation plan is available.

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes. The bedding should extend to the spring line of the pipe. 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 bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 98% of the SPMDD.

Generally, it should be possible to re-use the moist, not wet, brown silty clay above the cover material if the excavation and filling operations are carried out in dry weather conditions. The wet silty clay should be given a sufficient drying period to decrease its moisture content to an acceptable level to make compaction possible prior to being re-used.

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.

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 bearing surfaces and 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 Person as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters, tarpaulins or other suitable means.

In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms.

Precautions should be taken if activities are to be carried out during freezing conditions. Additional information could be provided, if required.

6.7 Corrosion Potential and Sulphate

One (1) sample was submitted for testing. The analytical test results of the soil sample indicate that the sulphate content is less than 0.01%. These results along with the chloride and pH value are indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the sample indicate 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

The proposed development is located in an area of low to medium sensitive silty clay deposits for tree planting. Based on our review of the subsurface profile below the subject site, the underlying silty clay deposit is relatively dry and designated as a very stiff to firm silty clay. Therefore, the proposed development is considered to be located within an area of low sensitive silty clay deposits for tree planting.

Tree Planting Restrictions

Based on the results of the representative soil samples, the subject site is considered as a **low/medium sensitivity** area for tree planting according to the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines).

Since the modified plasticity limit (PI) generally does not exceed 40%, large trees (mature height over 14 m) can be planted at the subject site 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).

Based on our testing results, tree planting setback limits should be 4.5 m for small (mature tree height up to 7.5m) and medium size trees (mature tree height 7.5 m to 14 m) provided that the following conditions are met:

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

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

- The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.

- The foundation walls are to be reinforced at least nominally (minimum of two upper and two lower 15M bars in the foundation wall).

- Grading surrounding the tree must promote drainage to the tree root zone (in such a manner as not to be detrimental to the tree).

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 of detailed grading, servicing, landscaping and structural plan(s) from a geotechnical perspective.

☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not by Paterson, prior to construction (if applicable).

☐ Review of architectural plans pertaining to groundwater suppression systems, 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 used.

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

☐ Observation of all subgrades prior to backfilling.

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

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

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soils, with the exception of engineered crushed stone fill, generated by construction activities that will be transported on-site or off-site should be handled as per *Ontario Regulation 406/19: On-Site and Excess Soil Management.*

8.0 Statement of Limitations

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

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

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine its 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 Ottawa Community Housing or their agents is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

O.Canton

Owen R. Canton, B.Eng.

LICENSED PROFESSIONAL ENGINEER
March 5, 2025
F. I. ABOU-SEIDO
100156744
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng.

Report Distribution:

☐ Ottawa Community Housing (email copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

SOIL PROFILE AND TEST DATA SHEETS BY OTHERS

ATTERBERG LIMITS TESTING RESULTS

GRAIN SIZE AND HYDROMETER TESTING RESULTS

ANALYTICAL TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

3380 Jockvale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364806.42 **NORTHING:** 5013924.35 **ELEVATION:** 91.65

PROJECT: Proposed Residential Development

BORINGS BY: Track Mounted Drill Rig

REMARKS:

DATE: January 30, 2025

FILE NO. : PG5676

HOLE NO. : BH 1-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Dense to compact, brown silty sand, with gravel, cobbles and crushed stone

1.22m [90.43m]

Compact to very dense, brown **SILTY SAND**, with gravel, cobble sand boulders

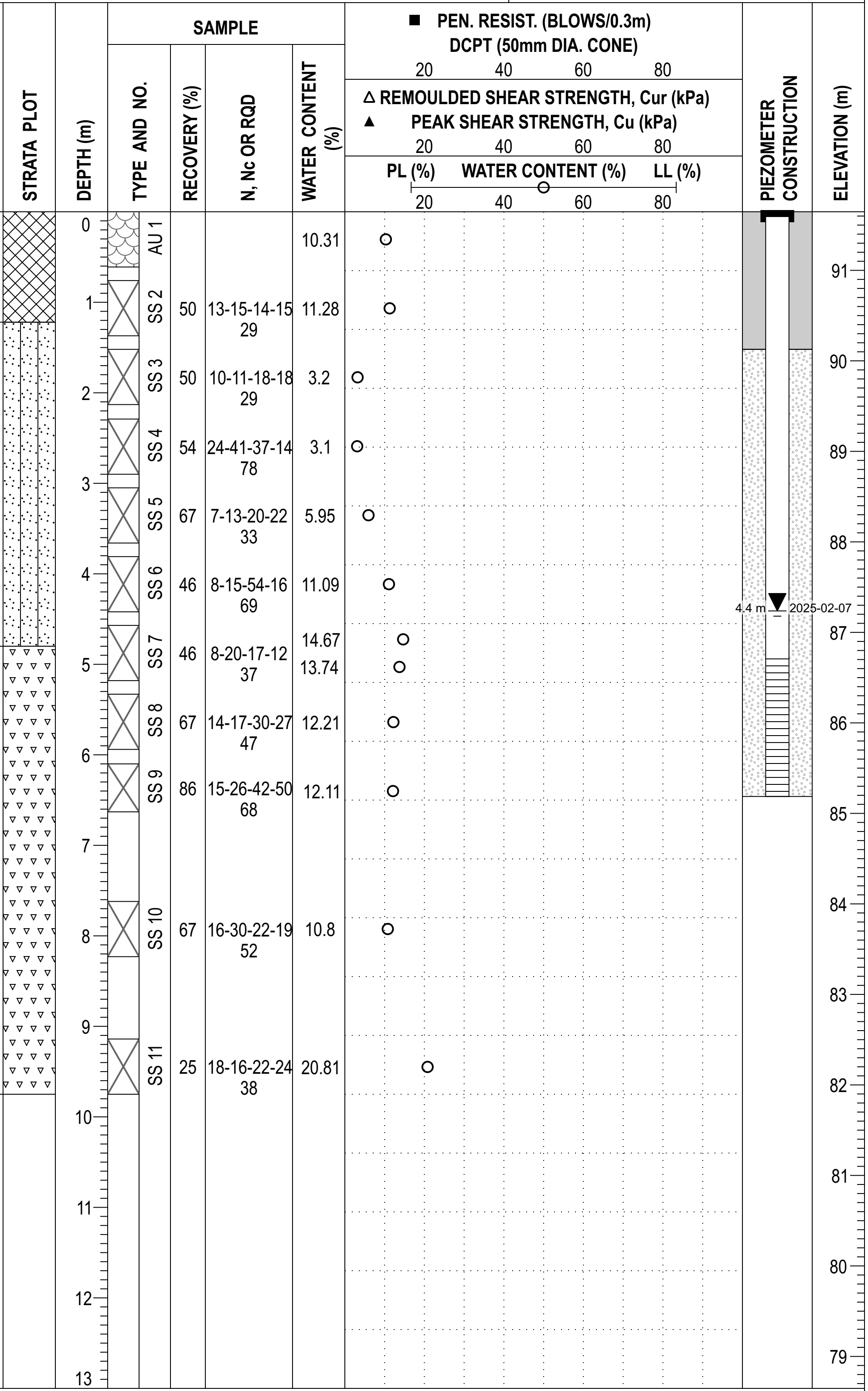
4.80m [86.85m]

GLACIAL TILL: Dense to very dense, brown silty sand, with gravel, cobbles and boulders
- Grey by 5.3 m depth

9.75m [81.90m]

End of Borehole

(GWL at 4.41 m depth - February 7, 2025)



SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			10.31
SS 2	50	13-15-14-15 29	11.28
SS 3	50	10-11-18-18 29	3.2
SS 4	54	24-41-37-14 78	3.1
SS 5	67	7-13-20-22 33	5.95
SS 6	46	8-15-54-16 69	11.09
SS 7	46	8-20-17-12 37	14.67 13.74
SS 8	67	14-17-30-27 47	12.21
SS 9	86	15-26-42-50 68	12.11
SS 10	67	16-30-22-19 52	10.8
SS 11	25	18-16-22-24 38	20.81

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

3380 Jockvale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364779.64 **NORTHING:** 5013943.84 **ELEVATION:** 91.77

PROJECT: Proposed Residential Development

BORINGS BY: Track Mounted Drill Rig

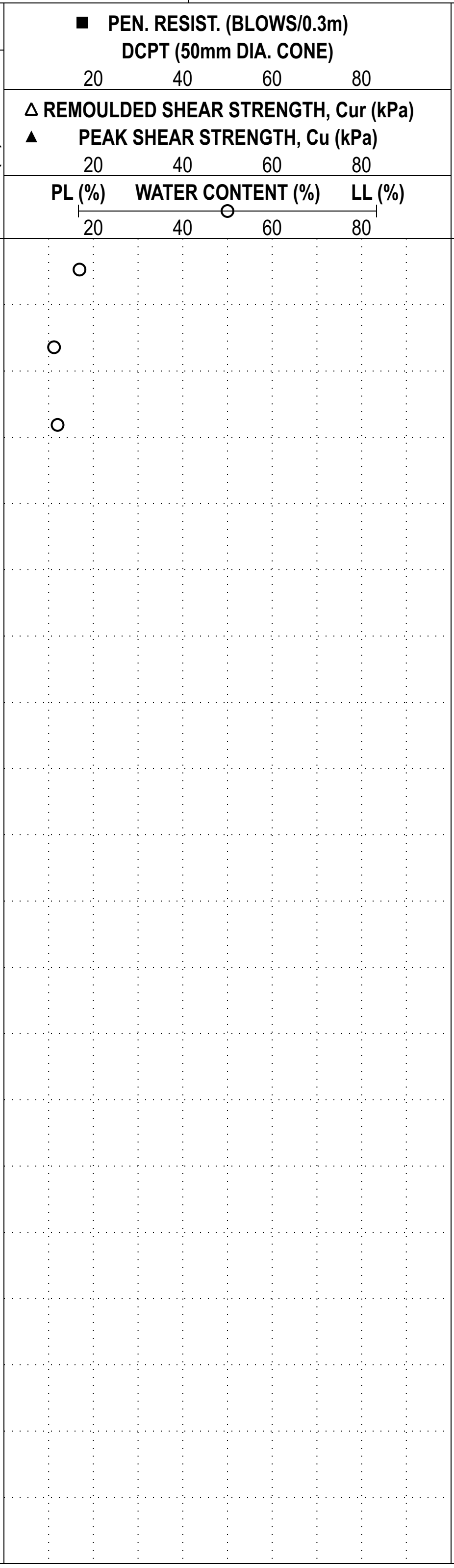
REMARKS:

DATE: January 30, 2025

FILE NO. : PG5676

HOLE NO. : BH 2-25

SAMPLE DESCRIPTION	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)	ELEVATION (m)
GROUND SURFACE	0					91.77
FILL: Brown silty sand, with gravel, cobbles and boulders, trace clay		AU 1			16.9	
	1	SS 2	17	6-13-10-12 23	11.21	91
1.92m [89.84m]	2	SS 3	50	20-10-10-7 20	12.0	90
End of Borehole Practical refusal to augering at 1.93 m depth						



PIEZOMETER CONSTRUCTION

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

3380 Jockvale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364779.64 **NORTHING:** 5013943.84 **ELEVATION:** 91.77

PROJECT: Proposed Residential Development

BORINGS BY: Track Mounted Drill Rig

REMARKS:

DATE: January 30, 2025

FILE NO. : PG5676

HOLE NO. : BH 2A-25

SAMPLE DESCRIPTION

GROUND SURFACE

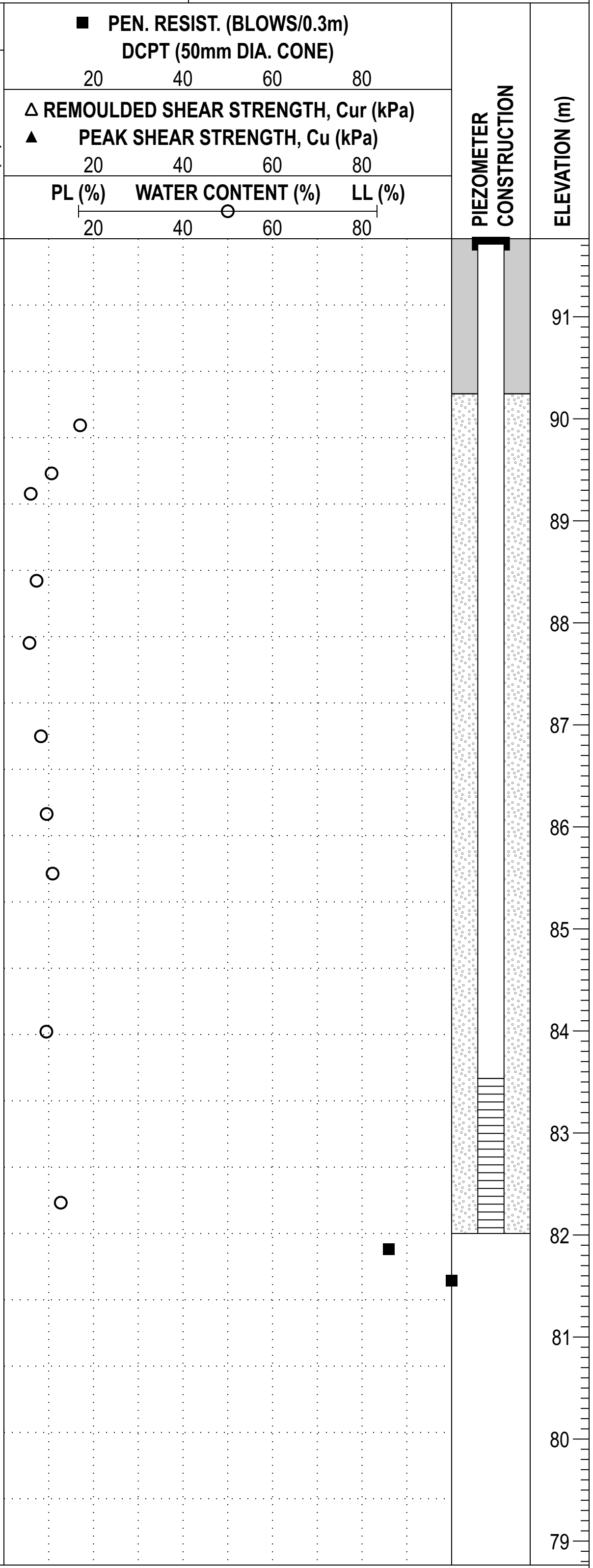
Depth to (m)	Sample Description
1.52m [90.25m]	Refer to BH 2-25 for soil stratigraphiy
2.44m [89.33m]	**FILL:** Brown silty sand, with gravel and cobbles, trace clay
4.32m [87.44m]	Dense to very dense, brown **SILTY SAND** with gravel, cobbles and boulders
9.75m [82.02m]	**GLACIAL TILL:** Very dense to dense, grey silty sand, with gravel, cobbles and boulders
10.13m [81.64m]	Dynamic Cone Penetration Test commenced at 9.75 m depth

End of Borehole

Practical refusal to DCPT at 10.13 m depth

(BH piezometer blocked - February 7, 2025)

TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
SS 1	54	6-5-14-13 19	17.02
SS 2	89	15-50-/-/ 50/0.05	10.64 5.99
SS 3	63	12-17-23-18 40	7.29
SS 4	44	14-26-50-/ 76/0.25	5.71
SS 5	67	16-21-28-28 49	8.27
SS 6	75	25-27-33-40 60	9.54
SS 7	100	30-50-/-/ 50/0.1	10.87
SS 8	100	32-50-/-/ 50/0.13	9.48
SS 9	71	18-16-23-38 39	12.7



SOIL PROFILE AND TEST DATA

Geotechnical Investigation

3380 Jockvale Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 364776.61 **NORTHING:** 5013960.90 **ELEVATION:** 91.76

PROJECT: Proposed Residential Development

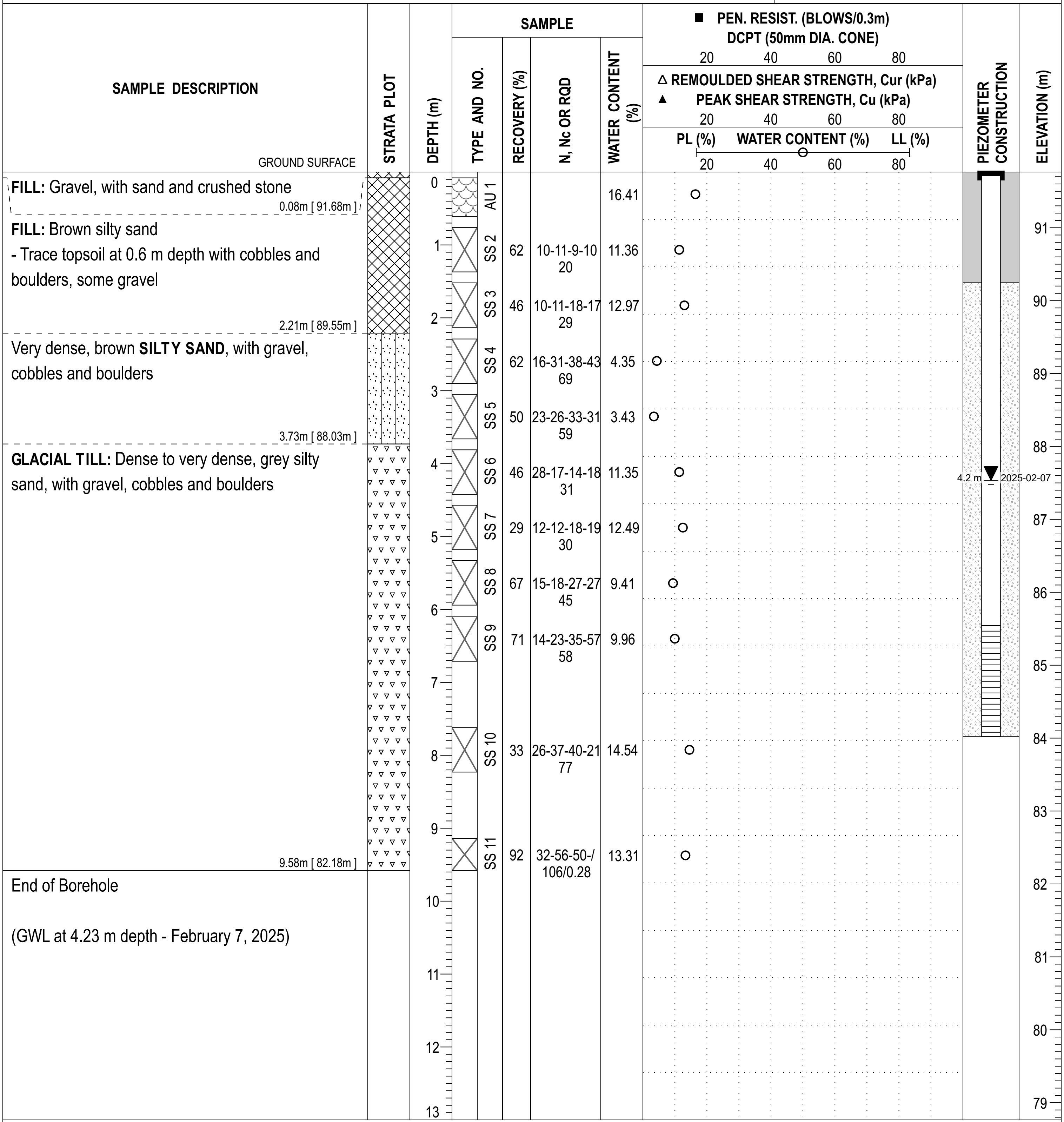
BORINGS BY: Track Mounted Drill Rig

REMARKS:

DATE: January 31, 2025

FILE NO. : PG5676

HOLE NO. : BH 3-25



SAMPLE DESCRIPTION	
GROUND SURFACE	
FILL: Gravel, with sand and crushed stone	0.08m [91.68m]
FILL: Brown silty sand - Trace topsoil at 0.6 m depth with cobbles and boulders, some gravel	2.21m [89.55m]
Very dense, brown **SILTY SAND**, with gravel, cobbles and boulders	3.73m [88.03m]
GLACIAL TILL: Dense to very dense, grey silty sand, with gravel, cobbles and boulders	9.58m [82.18m]
End of Borehole (GWL at 4.23 m depth - February 7, 2025)	

SAMPLE TYPE AND NO.	RECOVERY (%)	N, Nc OR RQD	WATER CONTENT (%)
AU 1			16.41
SS 2	62	10-11-9-10 20	11.36
SS 3	46	10-11-18-17 29	12.97
SS 4	62	16-31-38-43 69	4.35
SS 5	50	23-26-33-31 59	3.43
SS 6	46	28-17-14-18 31	11.35
SS 7	29	12-12-18-19 30	12.49
SS 8	67	15-18-27-27 45	9.41
SS 9	71	14-23-35-57 58	9.96
SS 10	33	26-37-40-21 77	14.54
SS 11	92	32-56-50-/ 106/0.28	13.31

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Development - 3380 Jockvale Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

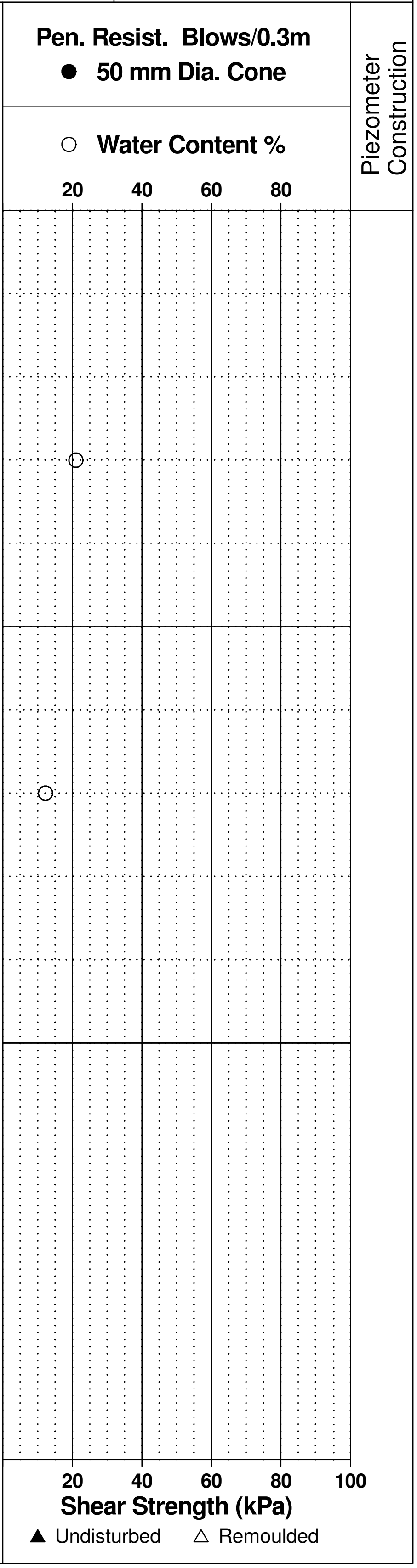
BORINGS BY Backhoe

DATE May 11, 2022

FILE NO. PG5676

HOLE NO. TP 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.50
FILL: Light brown silty sand with gravel, trace cobbles, crushed stone and organics		G	1				
0.30							
FILL: Brown silty clay with sand and gravel, trace crushed stone, cobbles, organics, and concrete		G	2				
0.90						1	91.50
GLACIAL TILL: Dense to very dense, brown silty sand with gravel, cobbles and boulders, trace clay		G	3				
2.00						2	90.50
End of Test Pit (TP dry upon completion)							



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Development - 3380 Jockvale Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

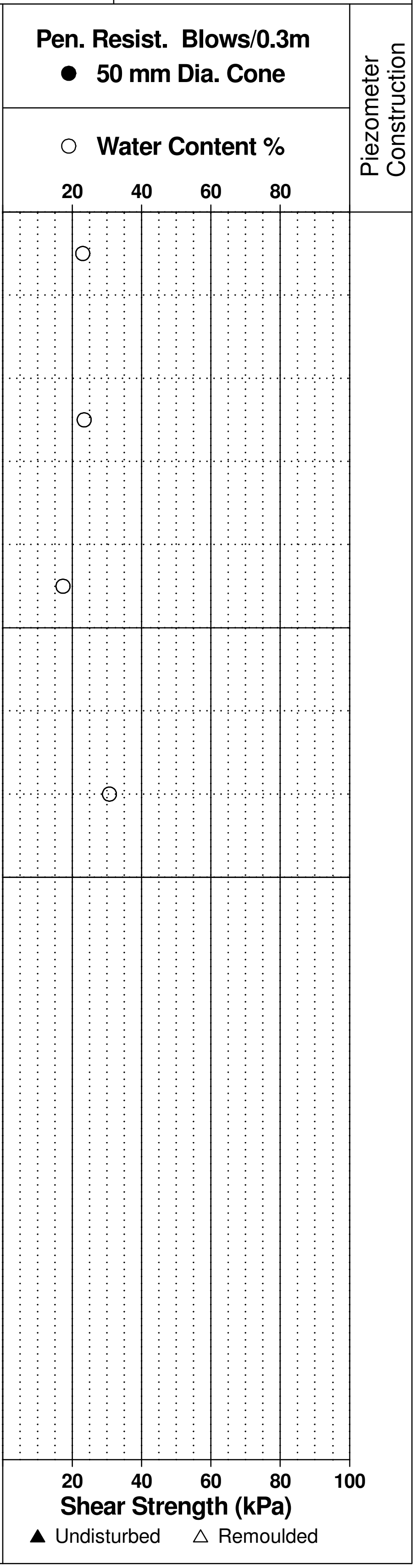
BORINGS BY Backhoe

DATE May 11, 2022

FILE NO. PG5676

HOLE NO. TP 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	91.91
TOPSOIL		G	1				
0.25							
GLACIAL TILL: Dense, brown silty clay with sand, some gravel, cobbles and boulders		G	2				
- grey to light brown by 0.75m depth		G	3				
						1	90.91
		G	4				
1.60							
End of Test Pit (TP dry upon completion)							



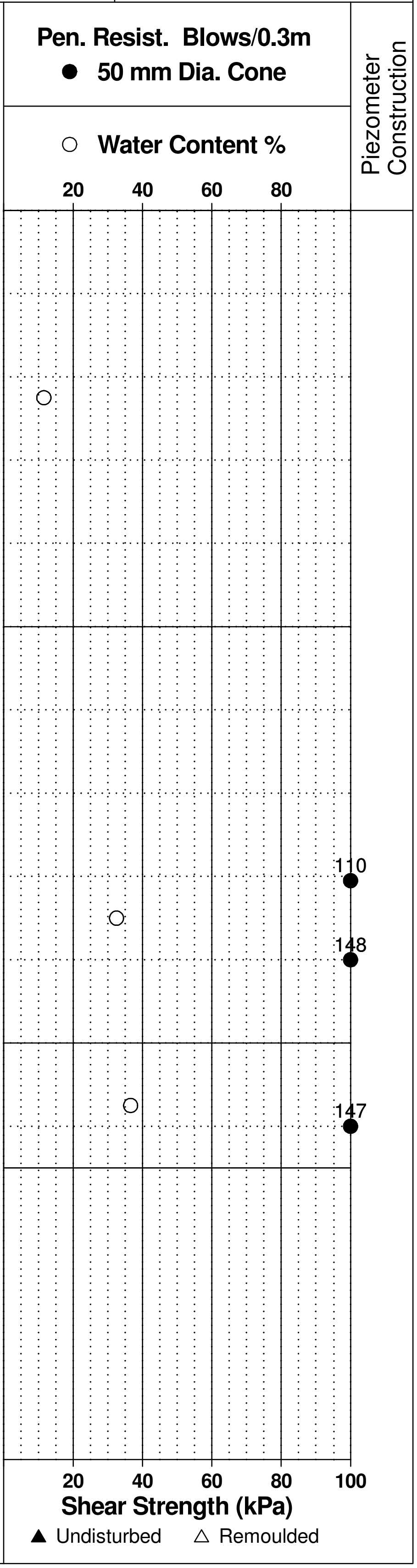
patersongroup Consulting Engineers
154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Development - 3380 Jockvale Road
Ottawa, Ontario

DATUM Geodetic
REMARKS
BORINGS BY Backhoe
DATE May 11, 2022
FILE NO. PG5676
HOLE NO. TP 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	92.41
FILL: Brown silty clay with sand and gravel, trace crushed stone, cobbles, topsoil and wood	▩	G	1				
						1	91.41
1.60							
Very stiff, brown **SILTY CLAY,** trace sand and organics	▨	G	2				
						2	90.41
		G	3				
2.30							
End of Test Pit (TP dry upon completion)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Supplemental Geotechnical Investigation
Proposed Development - 3380 Jockvale Road
Ottawa, Ontario

DATUM Geodetic

REMARKS

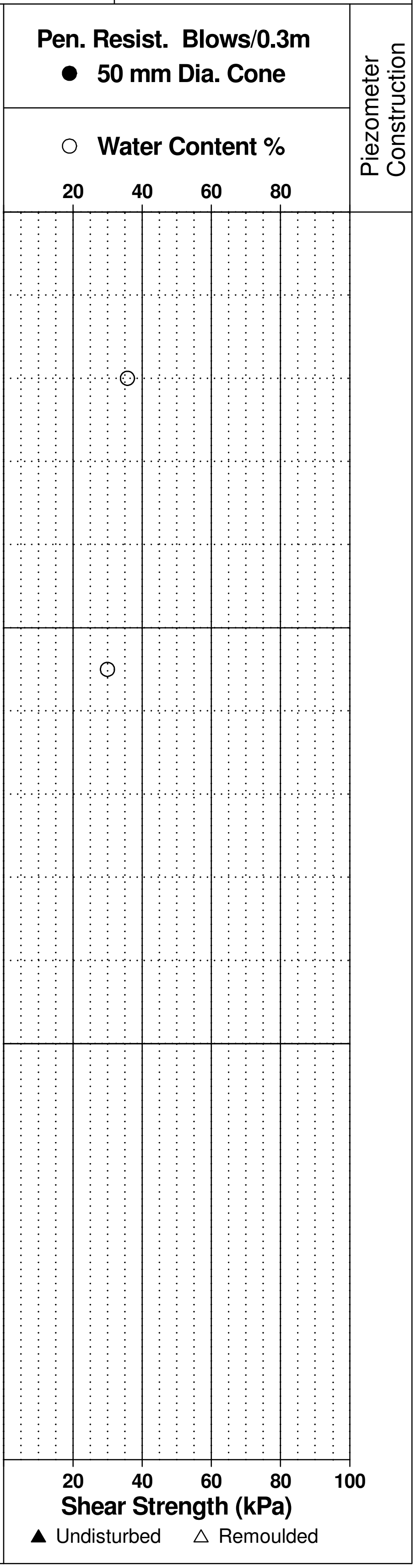
BORINGS BY Backhoe

DATE May 11, 2022

FILE NO. PG5676

HOLE NO. TP 4-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	91.50
FILL: Brown silty clay with sand, trace gravel, cobbles, boulders and topsoil		G	1				
0.75							
GLACIAL TILL: Very stiff, light brown silty clay with sand, gravel, cobbles and boulders		G	2			1	90.50
2.00						2	89.50
End of Test Pit (TP dry upon completion)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 3380 Jockvale Rd
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 6

FILE NO. PG5676

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	91.80
TOPSOIL 0.20						
FILL: Brown silty sand, some gravel and clay 0.91	AU	1				
GLACIAL TILL: Compact to very dense brown silty sand, with gravel, cobbles and frequent boulders	SS	2	33	12	1	90.80
	SS	3	50	+50	2	89.80
	SS	4	50	84		
					3	88.80
	SS	5	75	67		
	SS	6	21	38	4	87.80
	SS	7	38	23	5	86.80
5.92	SS	8	63	63		
End of Borehole (GWL @ 5.18 m depth - Jan 6, 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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Residential Development - 3380 Jockvale Rd
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5676**

REMARKS

HOLE NO. **BH 2-21**

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 6

SOIL DESCRIPTION GROUND SURFACE	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Piezometer Construction
TOPSOIL					0	91.96	
0.20 **FILL:** Brown silty clay to clayey silt, some sand and gravel	AU	1					
	SS	2	83	10	1	90.96	
1.82 **GLACIAL TILL:** Loose to compact brown silty sand, with gravel, cobbles and boulders	SS	3	0	10			
					2	89.96	∇
	SS	4	83	7			
					3	88.96	
	SS	5	79	7			
	SS	6	42	15	4	87.96	
4.87 **GLACIAL TILL:** Compact to dense grey silty sand, with gravel, cobbles and boulders	SS	7	21	+50	5	86.96	
	SS	8	42	18			
5.92 End of Borehole (GWL @ 2.13 m depth - Jan 6, 2021)							

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
Proposed Residential Development - 3380 Jockvale Rd
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE 2021 January 6

FILE NO. PG5676

HOLE NO. BH 3-21

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
FILL: Brown silty clay, some sand and gravel	AU	1		
0.61				
GLACIAL TILL: Compact brown silty sand, with gravel, cobbles and boulders	SS	2	79	36
	SS	3	21	3
	SS	4	29	9
	SS	5	8	8
3.65				
GLACIAL TILL: Compact to dense grey silty sand, some gravel and boulders	SS	6	21	8
	SS	7	38	18
	SS	8	63	39
5.92				
End of Borehole (GWL @ 3.81 m depth - Jan 6, 2021)				

DEPTH (m)	ELEV. (m)
0	91.38
1	90.38
2	89.38
3	88.38
4	87.38
5	86.38

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 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)

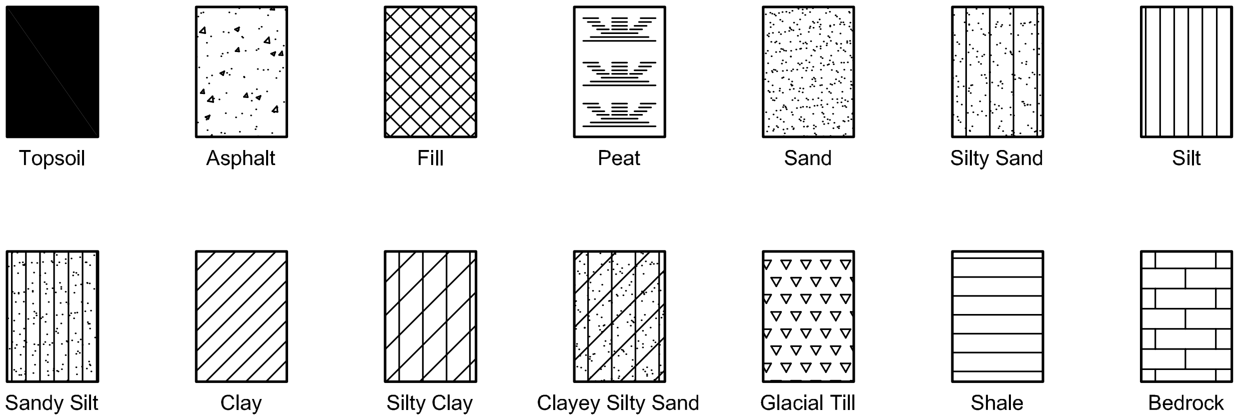
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

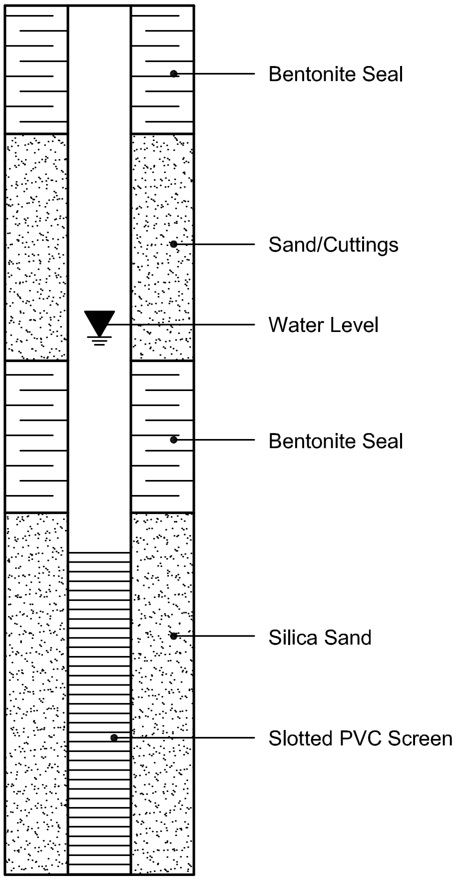
SYMBOLS AND TERMS (continued)

STRATA PLOT

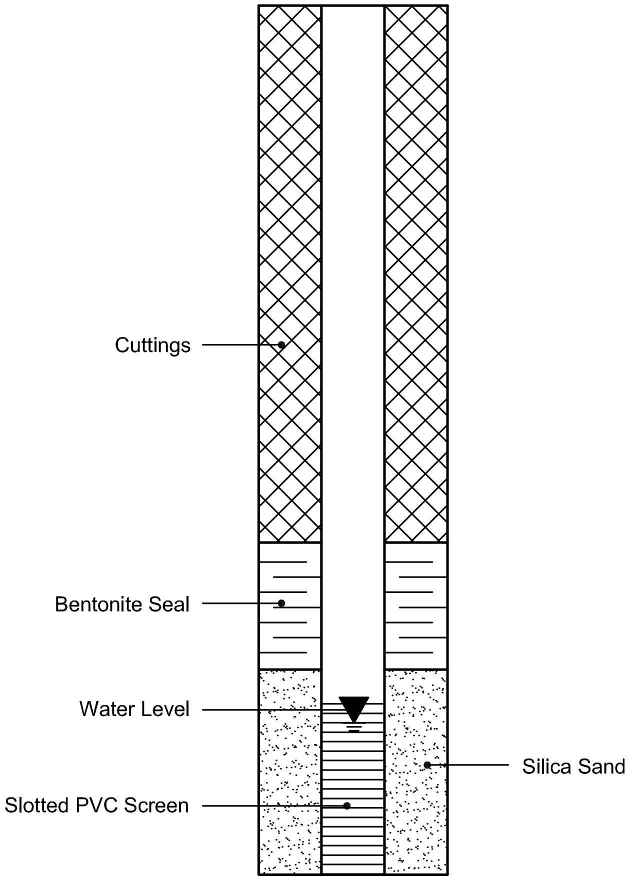


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



PROJECT: 1773927

RECORD OF BOREHOLE: 17-04

SHEET 1 OF 1

LOCATION: See Site Plan

BORING DATE: March 14, 2017

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-6} 10^{-5} 10^{-4} 10^{-3} WATER CONTENT PERCENT Wp ├──○W──┤ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	92.21							
0	Power Auger 200 mm Diam. (Hollow Stem)	(PT) Fibrous PEAT; dark brown; non-cohesive	0.00							
		(CI/CH) SILTY CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff	92.01 0.20	1	SS	6				
1				2	SS	8				
2		(SM) SILTY SAND, some gravel; brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist, compact	90.69 1.52	3	SS	15			MH	
				4	SS	13				
3		(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, compact	89.16 3.05	5	SS	15				▽
4		End of Borehole Auger Refusal	88.52 3.69							WL in open borehole at 3.20 m depth below ground surface upon completion of drilling
5										
6										
7										
8										
9										
10										

MIS-BHS 001 1773927.GPJ GAL-MIS.GDT 06/09/17 JM

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DG

CHECKED: TMS

PROJECT: 1773927

RECORD OF BOREHOLE: 17-05

SHEET 1 OF 1

LOCATION: See Site Plan

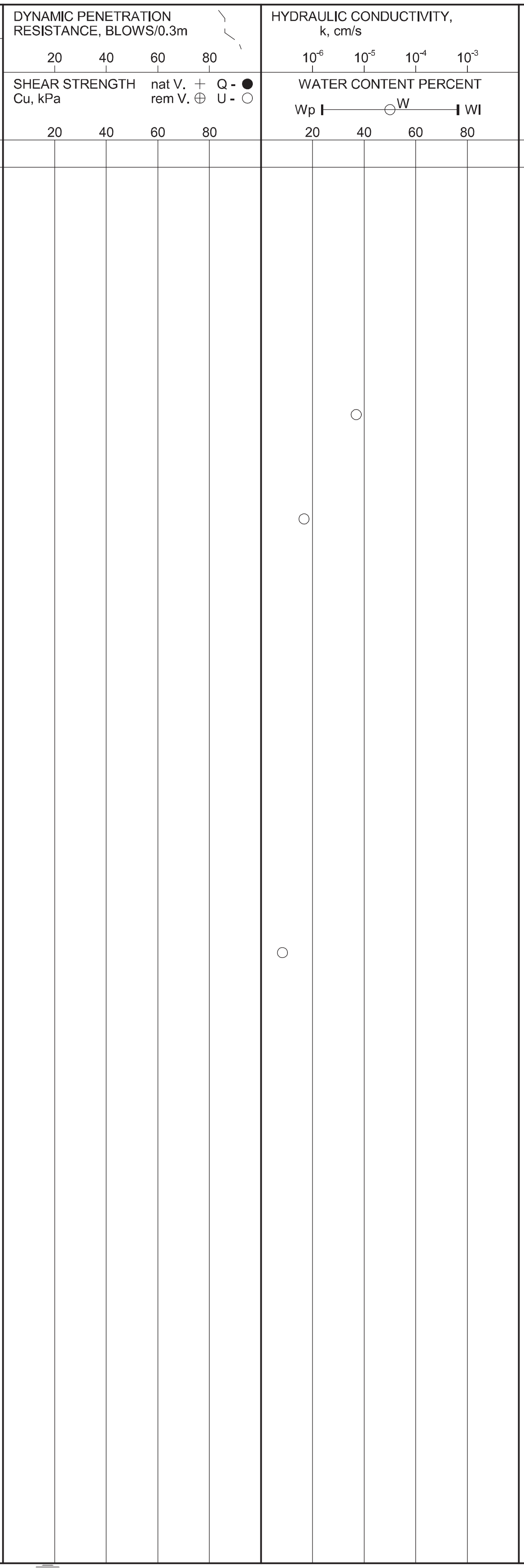
BORING DATE: March 14, 2017

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m
	GROUND SURFACE		91.44			
Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; dark brown; moist		0.00			
	(CI/CH) SILTY CLAY; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff		0.15	1	SS	5
				2	SS	5
				3	SS	3
	(SM) SILTY SAND, some gravel; brown, contains cobbles and boulders (GLACIAL TILL); non-cohesive, moist, loose		89.15 2.29	4	SS	7
	(SM) SILTY SAND, some gravel; grey, contains cobbles and boulders (GLACIAL TILL); non-cohesive, wet, very loose to very dense		88.09 3.35	5	SS	7
				6	SS	3
				7	SS	53
				8	SS	54
	End of Borehole		85.34 6.10			



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

WL in open borehole at 1.22 m depth below ground surface upon completion of drilling

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: DG

CHECKED: TMS

MIS-BHS 001 1773927.GPJ GAL-MIS.GDT 06/09/17 JM

PROJECT: 1523645

RECORD OF BOREHOLE: 15-2

SHEET 1 OF 2

LOCATION: N 5013925.3 ;E 364835.3

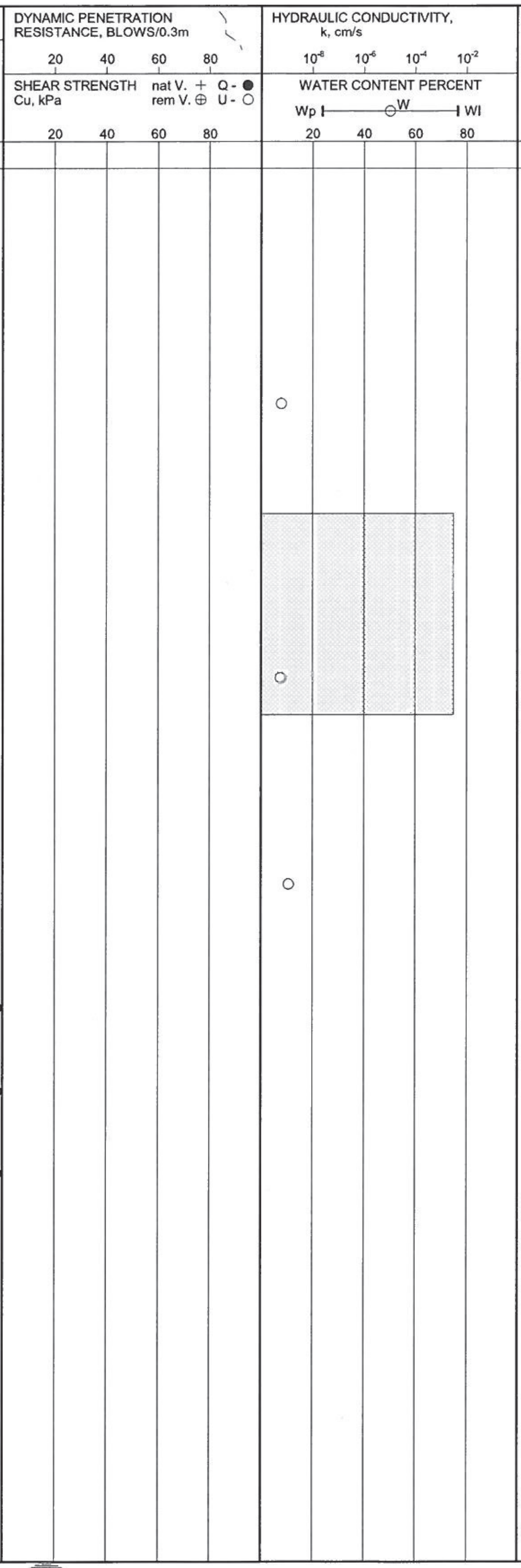
BORING DATE: August 14-17, 2015

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	91.59					
0	Power Auger 200 mm Diam. (Hollow Stem)	FILL - (SM) SILTY SAND, trace to some gravel, trace organic matter; dark brown; non-cohesive, moist		1	AS	-		Native Backfill
		TOPSOIL - (SM) SILTY SAND, trace gravel; dark brown; moist	0.10	2	AS	-		
1		(SM) gravelly SILTY SAND; grey brown, with oxidation staining, presence of cobbles and boulders inferred from auger resistance (GLACIAL TILL); non-cohesive, moist to wet, dense to compact	0.34	3	SS	39		Bentonite Seal
2				4	SS	18		Native Backfill ▽
				5	SS	25	MH	
3				6	SS	21		Bentonite Seal
4		(SW/GW and SM) SAND and GRAVEL and gravelly SILTY SAND; grey brown, interbedded, presence of cobbles and boulders inferred from observations in adjacent test pit excavations; non-cohesive, wet, compact	87.78 3.81	7	SS	16		Granitic Sand and Native Backfill ▽
5				8	SS	25		32 mm Diam. PVC #10 Slot Screen 'B'
				9	SS	26	M	Granitic Sand
6		(SM and SP) gravelly SILTY SAND and gravelly SAND; grey brown to grey, interbedded, presence of cobbles and boulders inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, dense to very dense	85.49 6.10	10	SS	38		Bentonite Seal
7				11	SS	45		Granitic Sand
8				12	SS	114	MH	
				13	SS	132		
9	Wash Boring NW Casing			14	SS	>120		Native Backfill
10				15	SS	>150		
				16	RC	DD		
11				17	SS	>250		
		Borehole continued on RECORD OF DRILLHOLE 15-2	80.01 11.58					
12								
13								
14								
15								



DEPTH SCALE

1 : 75

Golder Associates

LOGGED: PAH

CHECKED: SD

PROJECT: 1523645

LOCATION: N 5013925.3 ;E 364835.3

INCLINATION: -90° AZIMUTH: ---

RECORD OF DRILLHOLE: 15-2

DRILLING DATE: August 14-17, 2015

DRILL RIG: CME 850

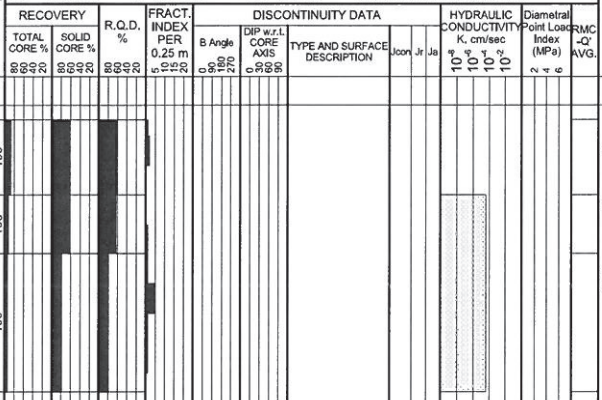
DRILLING CONTRACTOR: Marathon Drilling

SHEET 2 OF 2

DATUM: CGVD28

JN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	NOTE: For additional abbreviations refer to list of abbreviations & symbols.
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth	
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough	
CJ - Conjugate	CL - Cleavage	IR - Irregular	MB - Mechanical Break	

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN FLUSH
		BEDROCK SURFACE		80.01 11.58		
12	Rotary Drill NQ Core	Fresh, thinly to thickly bedded, grey DOLOMITE BEDROCK			1	100
13					2	100
14				77.06 14.53	3	100
15		End of Drillhole				
16						
17						
18						
19						
20						
21						
22						
23						
24						
25						
26						



WL in Screen 'A' at Elev. 87.49 m on Aug. 24, 2015

WL in Screen 'B' at Elev. 89.41 m on April 7, 2016

DEPTH SCALE

1 : 75

LOGGED: PAH

CHECKED: SD

PROJECT: 1523645

RECORD OF BOREHOLE: 15-3

SHEET 1 OF 1

LOCATION: N 5013871.5 ;E 364762.7

BORING DATE: August 13, 2015

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-8} 10^{-6} 10^{-4} 10^{-2} WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		91.39							
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (SM) SILTY SAND; dark brown; moist		0.00	1	AS	-				
1		(CI) sandy SILTY CLAY; grey brown, with oxidation staining, contains rootlets and silty sand seams (WEATHERED CRUST); cohesive, w>PL, very stiff		0.25	2	SS	6				
				89.41	3	SS	3				
2		(SM/SC) gravelly SILTY SAND to gravelly CLAYEY SAND; grey brown to grey, presence of cobbles and boulders inferred from auger resistance and auger refusal (GLACIAL TILL); non-cohesive, wet, compact to very loose to dense - Auger Refusal in Glacial Till at 2.97 m		1.98	4	SS	21				
3	Wash Boring NW Casing				5	SS	8				
4					6	SS	4				
5					7	SS	20			M	
				85.45	8	SS	45				
6		(SM) gravelly SILTY SAND; grey, presence of cobbles inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, very dense		5.94	9	SS	105				
7					10	SS	90				
8				83.16	11	SS	62				
		End of Borehole		8.23							
9											
10											
11											
12											
13											
14											
15											

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: PAH

CHECKED: SD

PROJECT: 1523645

RECORD OF BOREHOLE: 15-3A

SHEET 1 OF 1

LOCATION: N 5013899.3 ;E 364793.3

BORING DATE: August 17, 2015

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: Power Auger, 200 mm Diam. (Hollow Stem)

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

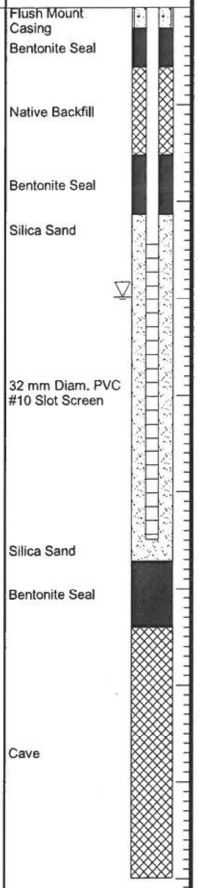
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-8} 10^{-6} 10^{-4} 10^{-2}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl : 20 40 60 80

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.30m	ADDITIONAL LAB. TESTING
GROUND SURFACE	91.34				
FILL - (SM) SILTY SAND; dark brown, contains organic matter; non-cohesive, moist	0.00				
	0.21	1	AS	-	
FILL - (SM/GM) SILTY SAND and GRAVEL; red brown and grey brown, contains organic matter; non-cohesive, moist, compact	90.63 0.71				
TOPSOIL - (ML) CLAYEY SILT; dark brown; moist	0.91	2	SS	11	
(CI) sandy SILTY CLAY; grey brown, with oxidation staining, contains rootlets and silty sand seams (WEATHERED CRUST); cohesive, w>PL, very stiff	89.51 1.83	3	SS	5	
(SM/SC) gravelly SILTY SAND to gravelly CLAYEY SAND; grey brown to grey, presence of cobbles and boulders inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, compact to very loose		4	SS	15	
		5	SS	2	MH
		6	SS	3	
(SM/GM) SILTY SAND and GRAVEL; grey, presence of cobbles inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, compact	86.77 4.57	7	SS	23	
		8	SS	15	
		9	SS	31	
(SM/GM and SP) SILTY SAND and GRAVEL and gravelly SAND; grey, interbedded, presence of cobbles and boulders inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, very dense	84.63 6.71	10	SS	67	
		11	SS	306	
End of Borehole	82.35 8.99				

PIEZOMETER OR STANDPIPE INSTALLATION



WL in Screen at Elev. 88.36 m on Aug. 24, 2015

DEPTH SCALE 1 : 75

LOGGED: PAH

CHECKED: SD

PROJECT: 1523645

RECORD OF BOREHOLE: 15-4

SHEET 1 OF 1

LOCATION: N 5013779.2 ;E 364691.4

BORING DATE: August 18, 2015

DATUM: CGVD28

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.30m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m 20 40 60 80 / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ 20 40 60 80	HYDRAULIC CONDUCTIVITY, k, cm/s 10^{-8} 10^{-6} 10^{-4} 10^{-2} / WATER CONTENT PERCENT Wp ⊢ ○W ⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE		92.00							
0	Power Auger 200 mm Diam. (Hollow Stem)	TOPSOIL - (ML) CLAYEY SILT; dark brown; moist		0.00 91.72	1	AS	-				
		(CI) SILTY CLAY, some sand; grey brown (WEATHERED CRUST); cohesive, w>PL, very stiff		0.28							
1					2	SS	9				
		(SC) gravelly CLAYEY SAND; grey brown to grey, presence of cobbles and boulders inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, loose to compact		90.55 1.45	3	SS	2				
2					4	SS	13				
3					5	SS	4				
4					6	SS	4				
		(SM/GM) SILTY SAND and GRAVEL; grey, presence of cobbles and boulders inferred from auger resistance and auger refusal (GLACIAL TILL); non-cohesive, wet, very dense		87.43 4.57	7	SS	65				
5					8	SS	>50				
		- 175 mm limestone cobble encountered at 5.79 m			9	RC	DD				
6		(ML/SM) sandy SILT to SILTY SAND, trace to some gravel; grey; non-cohesive, wet, dense to very dense		85.90 6.10	10	SS	42			M	
7											
8					11	SS	60				
		(SM) gravelly SILTY SAND; grey, contains sand layers, presence of cobbles and boulders inferred from auger resistance (GLACIAL TILL); non-cohesive, wet, very dense		83.31 8.69							
9					12	SS	>50				
10		End of Borehole		81.94 10.06							
11											
12											
13											
14											
15											

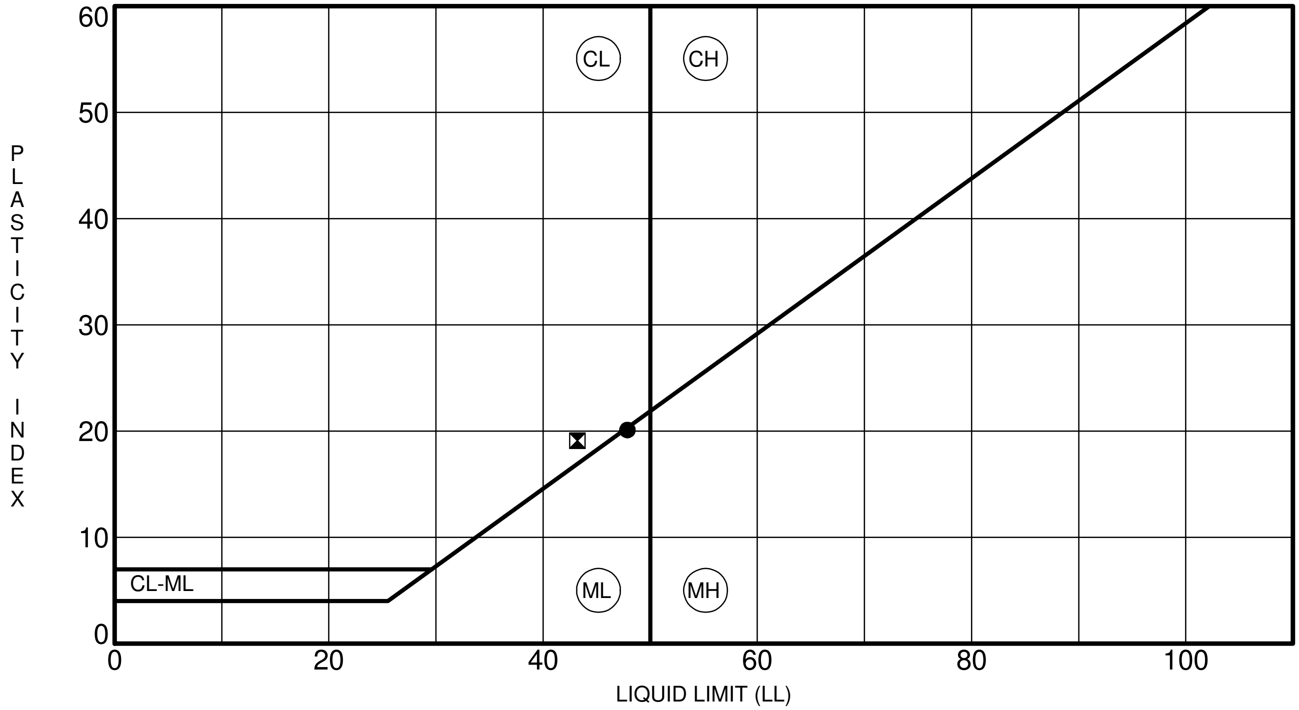
DEPTH SCALE

1 : 75

Golder Associates

LOGGED: PAH

CHECKED: SD



	Specimen Identification	LL	PL	PI	Fines	Classification
●	**TP 3-22 G3**	**48**	**28**	**20**		**CL - Inorganic clays of low plasticity**
⧗	**TP 3-22 G3**	**43**	**24**	**19**		**CL - Inorganic clays of low plasticity**

CLIENT **Lopers and Associates**

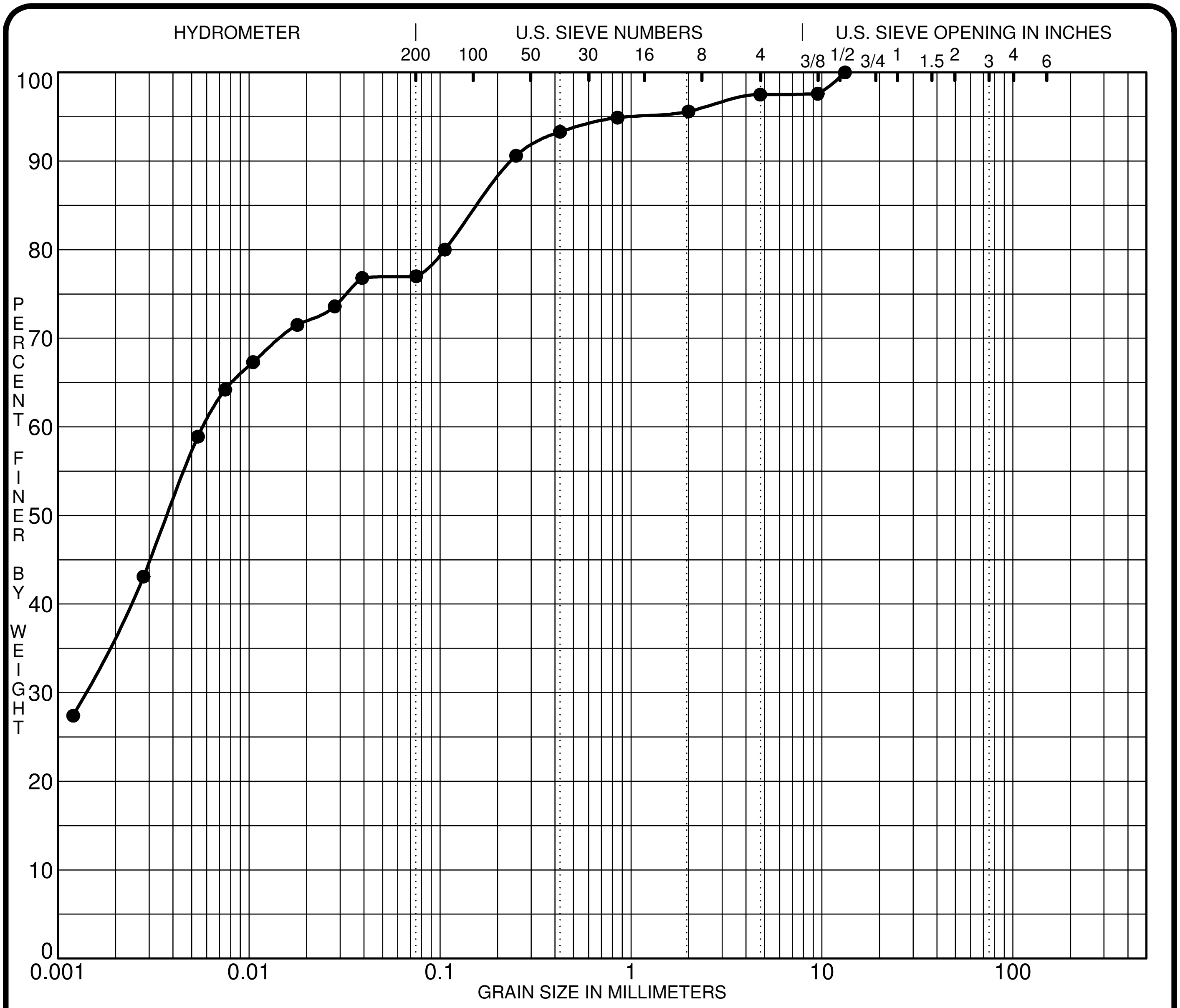
PROJECT **Supplemental Geotechnical Investigation -**
Proposed Development - 3380 Jockvale Road

FILE NO. **PG5676**

DATE **11 May 22**

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

ATTERBERG LIMITS' RESULTS



SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

	Specimen Identification		Classification	MC%	LL	PL	PI	Cc	Cu
●	**TP 3-22**	**G3**	**CL - Inorganic clays of low plasticity**						
☒									
▲									
★									

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**TP 3-22**	**G3**	**13.20**	**0.01**	**0.001**		**2.5**	**20.5**	**77.0**	
☒										
▲										
★										

CLIENT **Lopers and Associates**

FILE NO. **PG5676**

PROJECT **Supplemental Geotechnical Investigation -**
Proposed Development - 3380 Jockvale Road

DATE **11 May 22**

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

GRAIN SIZE DISTRIBUTION

Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 31263

Report Date: 13-Jan-2021

Order Date: 11-Jan-2021

Project Description: PG5676

	Client ID:	BH1-SS3	-	-	-
	Sample Date:	06-Jan-21 09:00	-	-	-
	Sample ID:	2103088-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	92.2	-	-	-
General Inorganics					
pH	0.05 pH Units	8.01	-	-	-
Resistivity	0.10 Ohm.m	73.4	-	-	-
Anions					
Chloride	5 ug/g dry	23	-	-	-
Sulphate	5 ug/g dry	21	-	-	-

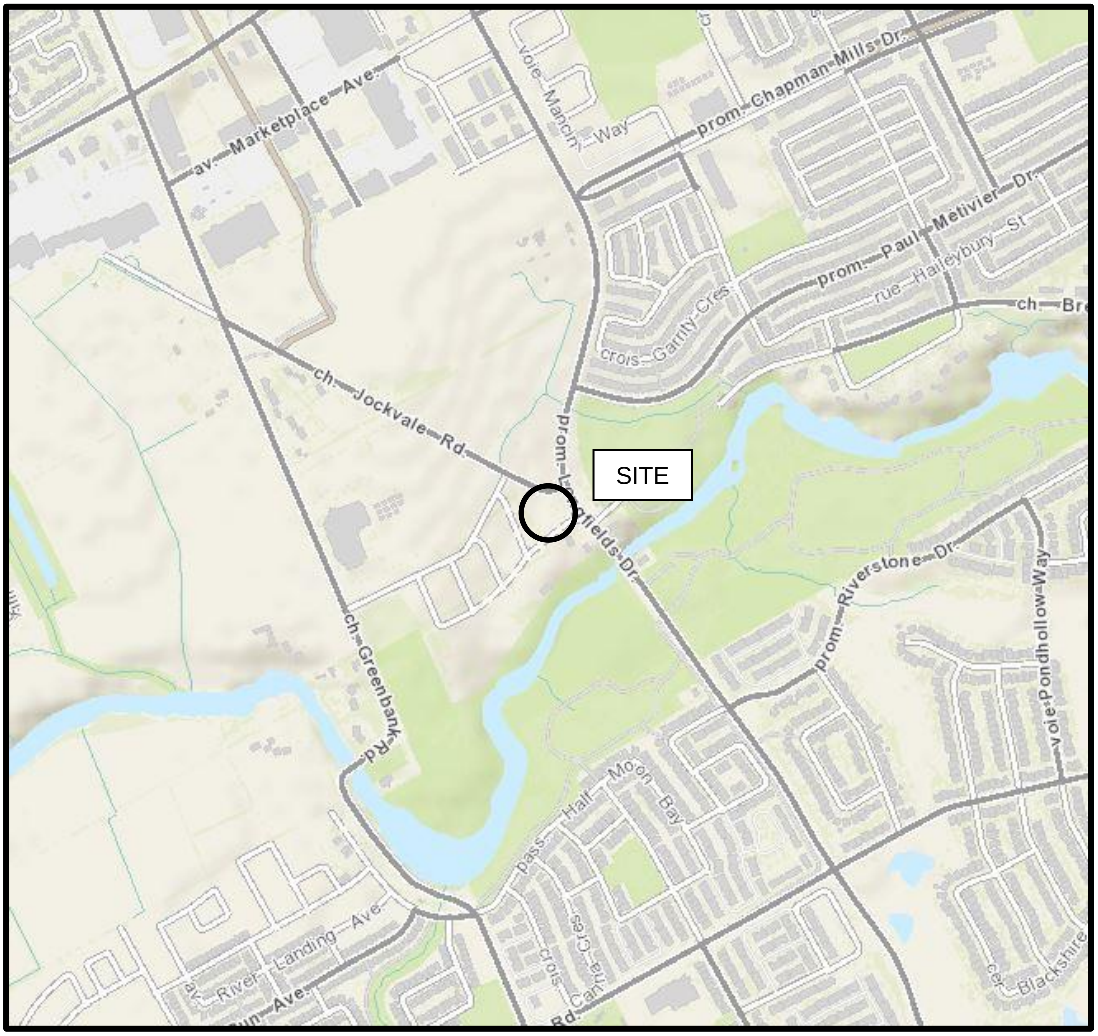
APPENDIX 2

FIGURE 1 – KEY PLAN

DRAWING PG5676-1 – TEST HOLE LOCATION PLAN

FIGURE 2 & 3 – SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5676-2 – PERMISSIBLE GRADE RAISE PLAN

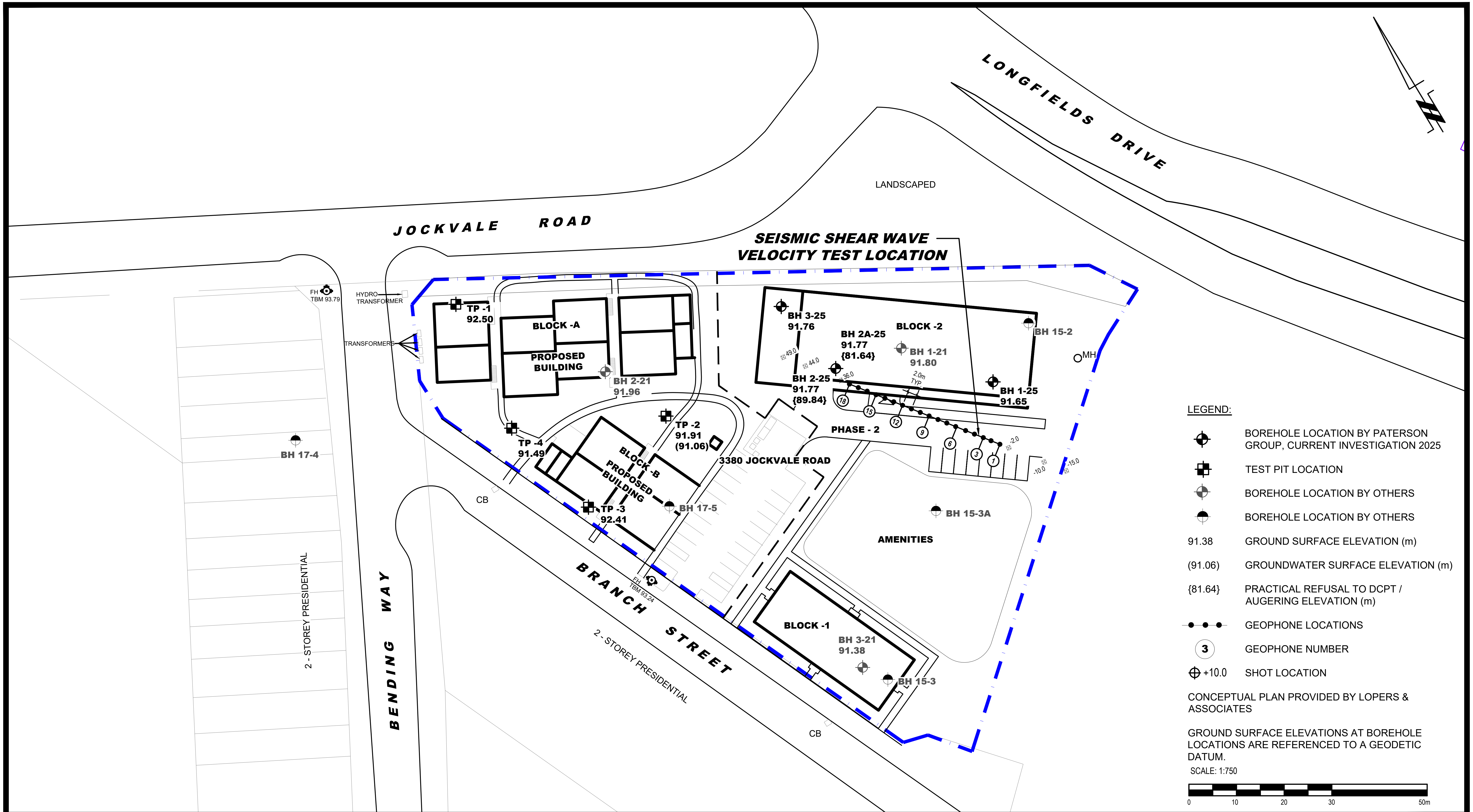


Source: GeoOttawa

FIGURE 1

KEY PLAN

PATERSON
GROUP



LEGEND:

⌖	BOREHOLE LOCATION BY PATERSON GROUP, CURRENT INVESTIGATION 2025
▣	TEST PIT LOCATION
⌖	BOREHOLE LOCATION BY OTHERS
◓	BOREHOLE LOCATION BY OTHERS
91.38	GROUND SURFACE ELEVATION (m)
(91.06)	GROUNDWATER SURFACE ELEVATION (m)
{81.64}	PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
●–●–●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⊕ +10.0	SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY LOPERS & ASSOCIATES

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

SCALE: 1:750

0 10 20 30 50m

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

NO.	REVISIONS	DATE	INITIAL
2	ADDED 2025 BOREHOLES AND SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	03/02/2025	YZ
1	REVISED WITH UPDATED CONCEPT PLAN	05/09/2024	OC

LOPERS & ASSOCIATES
SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
3380 JOCKVALE ROAD
OTTAWA, ONTARIO
Title:

TEST HOLE LOCATION PLAN

Scale: 1:750
Date: 10/2022
Drawn by: GK
Report No.: PG5676-1
Checked by: OC
Dwg. No.: PG5676-1
Approved by: FA
Revision No.: 2

11x17

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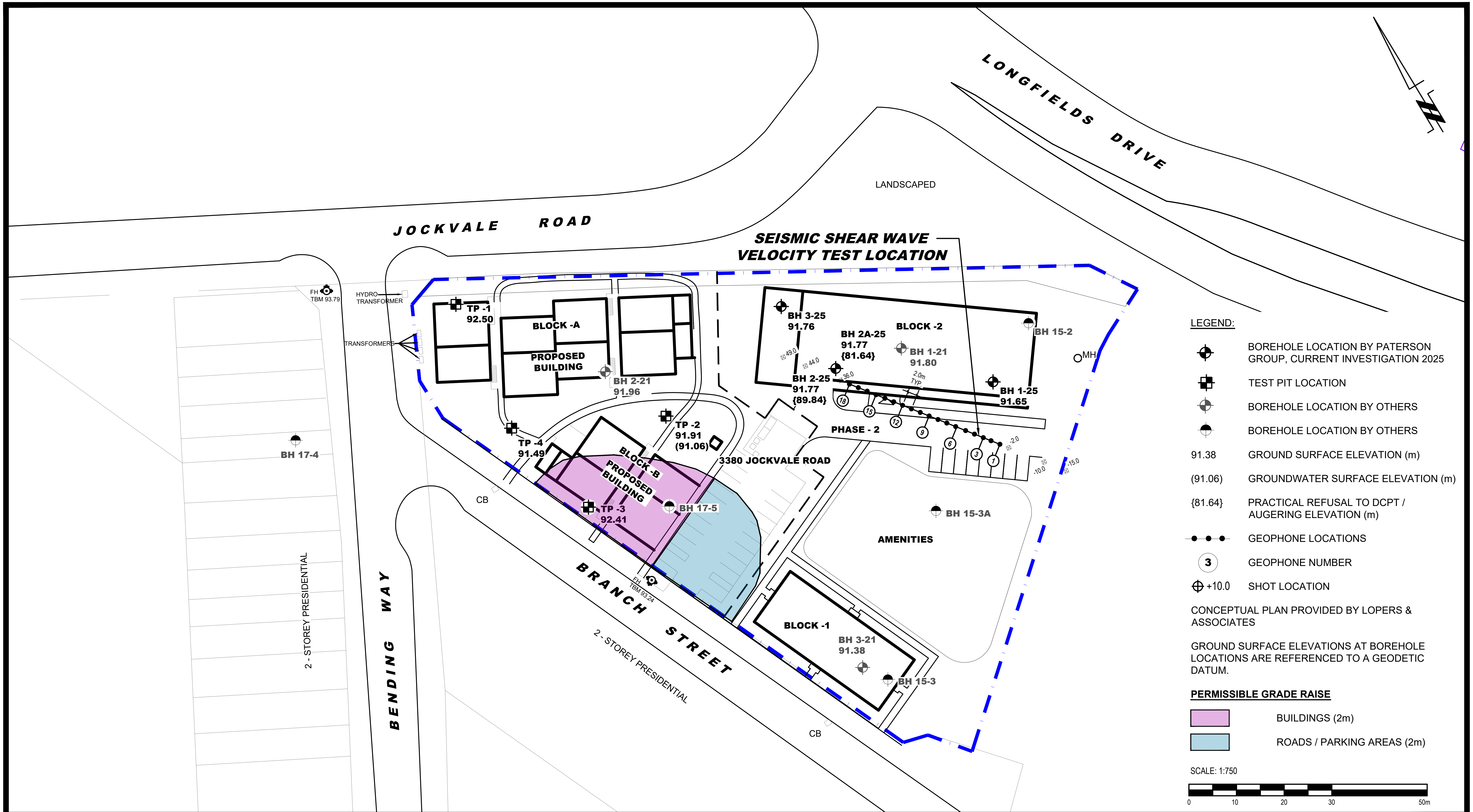

LEGEND:

⊕	BOREHOLE LOCATION BY PATERSON GROUP, CURRENT INVESTIGATION 2025
⊞	TEST PIT LOCATION
⊕	BOREHOLE LOCATION BY OTHERS
◓	BOREHOLE LOCATION BY OTHERS
91.38	GROUND SURFACE ELEVATION (m)
(91.06)	GROUNDWATER SURFACE ELEVATION (m)
{81.64}	PRACTICAL REFUSAL TO DCPT / AUGERING ELEVATION (m)
● ● ●	GEOPHONE LOCATIONS
③	GEOPHONE NUMBER
⊕ +10.0	SHOT LOCATION

CONCEPTUAL PLAN PROVIDED BY LOPERS & ASSOCIATES

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

PERMISSIBLE GRADE RAISE

🟪	BUILDINGS (2m)
🟦	ROADS / PARKING AREAS (2m)

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

NO.	REVISIONS	DATE	INITIAL
2	ADDED 2025 BOREHOLES AND SEISMIC SHEAR WAVE VELOCITY TEST LOCATION	14/02/2025	YZ
1	REVISED WITH UPDATED CONCEPT PLAN	05/09/2024	OC

LOPERS & ASSOCIATES

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
3380 JOCKVALE ROAD

OTTAWA, ONTARIO

Title:

PERMISSIBLE GRADE RAISE PLAN

Scale: 1:750	Date: 10/2022
Drawn by: GK	Report No.: PG5676-1
Checked by: OC	Dwg. No.: PG5676-2
Approved by: FA	Revision No.: 2

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