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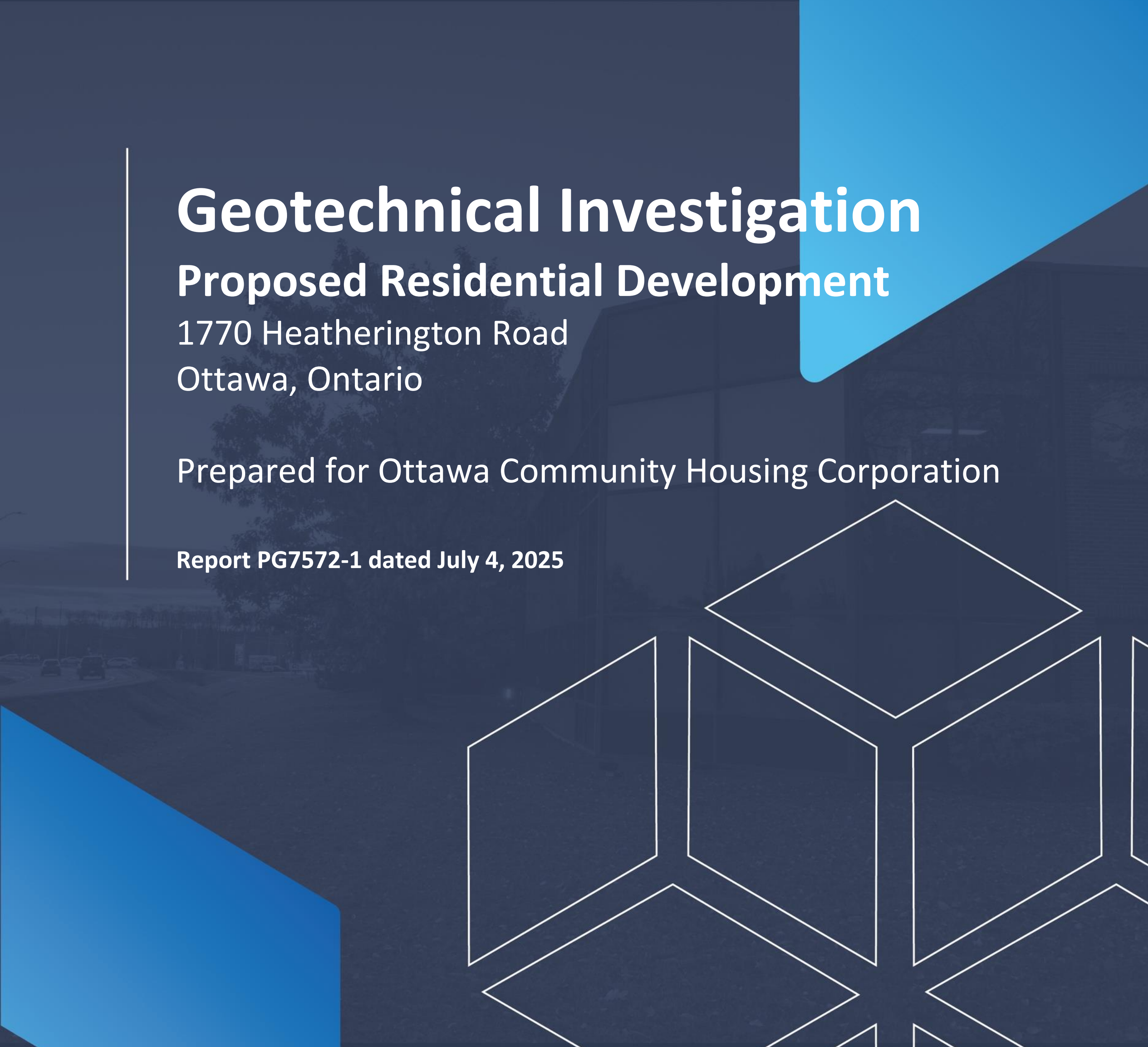

Geotechnical Investigation

Proposed Residential Development

1770 Heatherington Road
Ottawa, Ontario

Prepared for Ottawa Community Housing Corporation

Report PG7572-1 dated July 4, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by Ottawa Community Housing Corporation to conduct a geotechnical investigation for the proposed development to be located at 1770 Heatherington Road in the City of Ottawa (reference should be made 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 site was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

It is understood that the proposed development is anticipated to consist of two (2) four-storey residential buildings. The buildings are anticipated to consist of slab-on-grade construction. Associated local access roadways as well as parking and landscaped areas and are also anticipated as part of the subject development.

It is understood the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on June 5 and 6, 2025 and consisted of advancing a total of seven (7) boreholes to a maximum depth of 6.7 m below existing ground surface.

The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. Boreholes were advanced using a low-clearance rubber-track mounted drill rig operated by a two-person crew. The drilling procedure consisted of augering to the required depths at the selected locations and sampling the overburden soils. The fieldwork was conducted under the full-time supervision of our personnel under the direction of a senior engineer from our geotechnical department.

Previous investigations were conducted at the subject site by others in 2024, 2023, and 2008, and consisted of advancing a total of three (3), two (2), and one (1) boreholes within the subject site parcels to a maximum depth of 9.9 m.

The test hole locations from the current and previous investigations are shown on Drawing PG7572-1 - Test Hole Location Plan included in Appendix 2.

Sampling and In Situ Testing

Soil samples were collected from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler during the drilling program. All soil samples were classified on site and placed in sealed plastic bags.

All soil samples were transported to our laboratory for further examination. The depths at which the auger and split-spoon samples were recovered from the test holes are shown as AU and SS, respectively, on the Soil Profile and Test Data sheets in Appendix 1.

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

Undrained shear strength testing was carried out in cohesive soils using a field vane apparatus.

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

Groundwater

Groundwater monitoring wells were installed at BH 6-25 and BH 7-25, while the remaining boreholes were fitted with a flexible polyethylene standpipe. Monitoring wells and standpipes were installed to allow groundwater level monitoring subsequent to the completion of the field program.

Typical monitoring well construction details are described below:

- Slotted 51-mm diameter PVC screen at the base of the aforementioned boreholes.
- 51-mm diameter PVC riser pipe from the top of the screen to the ground surface.
- No. 3 silica sand backfill within annular space around screen.
- 300 mm thick bentonite hole plug directly above PVC slotted screen.
- Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

The groundwater level readings were obtained after a suitable stabilization period following the completion of the field investigation. Groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a high precision handheld GPS and referenced to a geodetic datum.

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. Select soil samples recovered from the subject site were submitted for Atterberg limit testing and grain size distribution testing.

3.4 Analytical Testing

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

3.5 Hydraulic Conductivity Testing

Hydraulic conductivity testing was conducted at one (1) monitoring well location to provide insight on the hydraulic properties of the bedrock at the subject site. The test data was analyzed using AQTESOLV Pro Version 4.5 aquifer analysis software package by HydroSOLVE Inc and the results were processed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous aquifer of infinite extent and a screen length significantly greater than the monitoring well diameter.

The assumption regarding aquifer storage is considered to be appropriate for groundwater inflow through the overburden aquifer. The assumption regarding screen length and well diameter is considered to be met based on a screen length of 1.5 m and a diameter of 0.05 m. While the idealized assumptions regarding aquifer extent and homogeneity are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. The testing results are further discussed in Subsection 4.4 of this report.

4.0 Observations

4.1 Surface Conditions

The overall development site consists of vacant land with an existing one-storey youth center building located within the east portion of the site. The subject parcels are located to the north and south of the existing building. The subject Building 1 is proposed within the northeastern parcel, and the subject Building 14 is proposed within the southeastern parcel. The site was observed to be relatively flat and approximately at grade with adjacent roadways and neighboring properties. The geodetic ground surface elevation within the northern parcel was measured to be approximately 87.5 m, while the geodetic ground surface elevation within the southern parcel was measured to be approximately 86.6 m.

The northern parcel is bordered to the north by existing commercial buildings, to the west by vacant land, to the south by a future roadway and further by an existing youth center building, and to the east by Heatherington Road. The southern parcel is bordered to the north by a future roadway and further by an existing youth center building, to the west by vacant land, to the south by an existing residential development, and to the east by Heatherington Road.

4.2 Subsurface Profile

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

Northern Parcel – Building 1

Generally, the subsurface profile at the test hole locations within the northern parcel consists of a layer of fill underlain by a deposit of glacial till, and further by the underlying bedrock formation.

The fill layer was generally observed to consist of a blend of silty sand and crushed stone. The fill layer was noted to extend to approximate depths between 3 to 3.4 m below the existing ground surface. The glacial till deposit was generally observed to consist of compact to dense brown or grey silty sand with variable amounts of clay, gravel, cobbles, and boulders. The upper 2.3 m of the glacial till deposit encountered within borehole BH 3-25 was noted to consist of stiff, brown silty clay with variable amounts of sand, gravel, cobbles, and boulders. It should be noted that a localized area of stiff to firm silty clay was encountered during the previous 2008 investigation by Others, in which the silty clay layer was measured to extent between a depth of 1.4 and 3 m below the existing ground surface at the time of the investigation.

Southern Parcel – Building 14

Generally, the subsurface profile at the test hole locations within the southern parcel consists of a layer of fill underlain by a deposit of silty clay, further underlain by a deposit of glacial till, followed by the underlying bedrock formation.

The fill layer was generally observed to consist of a blend of silty sand or silty clay with variable amounts of topsoil, organics, and crushed stone. The fill layer was noted to extend to an approximate depth of 0.7 m below the existing ground surface. The very stiff to stiff brown silty clay deposit was noted to extend to an approximate depth of 2.3 m below the existing ground surface. The stiff to firm grey silty clay deposit was noted to extend to approximate depths ranging between 3.7 and 4.5 m below the existing ground surface.

The upper layer of the glacial till deposit was observed to consist of stiff grey silty clay with variable amounts of sand, gravel, cobbles, and boulders, and was noted to extend to approximate depths ranging between 4.5 and 5.3 m below the existing ground surface. The lower layer of the glacial till deposit was observed to consist of compact to dense brown or grey silty sand with variable amounts of clay, gravel, cobbles, and boulders, and was noted to extend to approximate depths ranging between 4.5 and 5.3 m below the existing ground surface.

Bedrock

Based on available geological mapping, bedrock in the area of the subject site consists of shale with interbedded limestone of the Carlsbad Formation with a drift thickness ranging between 5 to 10 m.

Atterberg Limits Testing

Atterberg limits testing was completed on select samples recovered by Paterson and by others throughout the subject site during the current and previous investigations, respectively. The results of the Atterberg Limits testing are presented in Table 1 and on the Atterberg Limits Results sheet in Appendix 1.

Table 1 – Atterberg Limits Results							
Borehole	**Sample**	**Depth (m)**	**LL (%)**	**PL (%)**	**PI (%)**	**w (%)**	**Classification**
Atterberg Limits Testing Results Based on the Current Investigation							
BH 4-25	SS6	3.8 – 4.4	26	15	11	22	CL
BH 6-25	SS6	3.8 – 4.4	23	15	8	20	CL
BH 7-25	SS4	2.3 – 2.9	73	34	39	68	CH
Atterberg Limits Testing Results Based on the 2024 Investigation by Others							
BH23-9	SS2	0.8 – 1.4	52	22	30	35	CH
BH24-14	SS5	3.0 – 3.5	42	16	26	55	CL
BH23-9	SS5	4.6 – 5.2	-	-	-	9	Non-Plastic
Notes: LL: Liquid Limit; PL: Plastic Limit; PI: Plastic Index; w: Moisture Content; CL: Inorganic Clay of Low Plasticity; CH: Inorganic Clay of High Plasticity							

Grain Size Distribution and Hydrometer Testing

Grain size distribution (sieve and hydrometer analysis) was completed by Paterson and by others on selected soil samples during the current and previous investigations, respectively. The results of the grain size analysis are summarized in Table 2 and presented on the Grain-Size Distribution and Hydrometer Testing Results sheets in Appendix 1.

Table 2 – Summary of Grain Size Distribution Analysis					
Sample	**Depth (m)**	**Gravel (%)**	**Sand (%)**	**Silt (%)**	**Clay (%)**
Grain Size Distribution Analysis Results Based on the Current Investigation					
BH 3-25 (SS5)	3.0 – 3.6	24.8	38.0	26.4	10.7
BH 3-25 (SS6)	3.8 – 4.4	21.9	43.9	31.4	2.8
BH 4-25 (SS6)	3.8 – 4.4	10.2	29.2	36.9	23.7
BH 6-25 (SS7)	4.6 – 5.2	11.8	45.2	39.7	3.3
BH 7-25 (SS7)	4.6 – 5.2	11.1	44.9	31.1	12.9
Grain Size Distribution Analysis Based on the 2024 Investigation by Others					
BH23-9 (SS2)	0.8 – 1.4	0	17	21	62
BH24-14 (SS5)	3.0 – 3.5	0	7	40	53
BH23-9 (SS5)	4.6 – 5.2	22	46	27	5

4.3 Groundwater

Groundwater levels were measured in the standpipe piezometers and monitoring wells on June 19, 2025 for the current investigation. The remaining monitoring wells installed as part of the previous investigation were measured as well. The measured groundwater levels are presented on the Soil Profile and Test Data sheets in Appendix 1, and in Table 3 below.

Table 3 – Measured Groundwater Levels

Test Hole Number	**Method**	**Ground Surface Elevation (m)**	**Measured Groundwater Level**		**Date**
			Depth (m)	**Elevation (m)**	
Groundwater Levels Based on the Current Investigation					
BH 1-25	Piezometer	87.45	Blocked	-	June 19, 2025
BH 2-25	Piezometer	87.65	1.88	85.77	
BH 3-25	Piezometer	87.50	Blocked	-	
BH 4-25	Piezometer	86.63	1.64	84.99	
BH 5-25	Piezometer	86.65	2.18	84.47	
BH 6-25	Monitoring Well	86.71	1.53	85.18	
BH 7-25	Monitoring Well	86.66	1.54	85.12	June 19, 2025
Groundwater Levels from Monitoring Wells Previously Investigations by Others					
MW08-10	Monitoring Well	87.1	1.31	85.79	June 19, 2025
MW08-14	Monitoring Well	88.15	1.90	86.25	
MW08-17	Monitoring Well	87.9	1.90	86.0	
NOTE: The ground surface elevations at the test hole locations was referenced to a geodetic datum and as surveyed by others.					

It should be noted that surface water can become trapped within a backfilled borehole column, which can lead to higher-than-normal groundwater level readings. Additionally, it should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, groundwater levels could vary at the time of construction.

4.4 Hydraulic Conductivity Testing Results

Hydraulic conductivity tests were conducted at one (1) monitoring well location at the subject site on June 19, 2025. The testing results are summarized in Table 4 below.

Table 4 – Summary of Hydraulic Conductivity Testing Results.

Test Hole ID	Ground Surface Elevation (m)	Testing Depth Interval (m)	Testing Elevation Interval (m)	K (m/s)	Test Type	Soil Type
BH 6-25	86.71	4.42-5.92	82.29-80.79	1.09×10^{-6}	Falling Head	Glacial Till
				1.36×10^{-6}	Rising Head	

Summary of Results

Hydraulic conductivity testing conducted at a monitoring well screened within the glacial till yielded a hydraulic conductivity values ranging from 1.09×10^{-6} to 1.36×10^{-6} m/s.

These values are generally consistent with typical published values for glacial till. It should be noted that hydraulic conductivity may vary across the subject site depending on the composition and compaction at a given location of the overburden.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. Based on the results of the field investigation, the proposed building (Building 1) within the northern site parcel may be founded on conventional spread footings placed on the in-situ and approved fill bearing surface, while the proposed building (Building 14) within the southern site parcel may be founded on conventional spread footings placed on the in-situ, undisturbed, very stiff silty clay bearing surface.

Consideration may be given to the use of a raft foundation or a deep foundation solution, such as end-bearing piles extending to the bedrock surface or rock-socketed pile piles, should the provided bearing resistance values for conventional spread footings be insufficient to support design building loads, as discussed in Subsection 5.3

Due to the presence of a silty clay layer, the subject site is subjected to a permissible grade restriction. Our permissible grade raise recommendations are discussed in Subsection 5.3.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and fill, such as those containing organic or deleterious materials, should be stripped from under any buildings and other settlement sensitive structures. Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

Existing construction remnants, such as foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants, such as foundation walls, should be excavated to a minimum of 1 m below finished grade.

Northern Parcel – Building 1

Existing fill, free of organic or deleterious materials, reviewed and approved by Paterson field personnel at the time of construction, may be left in place as subgrade for paved areas and for the proposed building's floor slab. Where crushed stone fill is encountered below footings for the proposed building, it is recommended to be sub-excavated a minimum depth of 500 mm below the design founding elevation of the footing. Localized portions of the fill layer that contain high amounts of organics or deleterious materials at the sub-excavation depth will be requested to be further removed and segregated from the in-situ fill layer.

The sub-excavated in-situ fill surface would be reviewed at the time of sub-excavating the subgrade surface to 500 mm below the design founding elevation and be proof-rolled (i.e., re-compacted) with a suitably sized vibratory sheepsfoot roller **under dry and above-freezing conditions** making several passes (i.e. a minimum of 5 to 6 passes) under the supervision of Paterson field personnel. A smooth drum roller should be considered for areas where the fill consists of sandy silt and/or silty sand with gravel and a negligible amount of clay content.

Provided the fill material is proof-rolled to the satisfaction of Paterson field personnel, and localized soft spots are removed and in-filled with approved fill, such as Ontario Provincial Standard Specifications (OPSS) Granular B Type II crushed stone, the in-situ and existing fill may remain in place to support the above-noted settlement sensitive structure.

Upon approval for reuse by Paterson field personnel, the sub-excavated in-situ crushed stone fill material may be reinstated in maximum 300 mm lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD). If lesser-suitable fill is observed by Paterson field personnel during field reviews, the sub-excavation should be reinstated with OPSS Granular B Type II crushed stone with a minimum 300 mm thick OPSS Granular A crushed stone cap, both compacted to 98% of the material's SPMDD.

If native in-situ very stiff to stiff silty clay is encountered at or within the sub-excavation below the underside of footing elevation, the sub-excavation, if undertaken, may be terminated at the silty clay bearing surface. Further, sub-excavation beyond this native silty clay overburden is not required. The silty clay bearing surface should then be reviewed and approved by Paterson field personnel prior to the placement of footings.

Fill Placement

Fill placed for grading beneath the building footprint should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A or Granular B Type II. The imported fill material should be tested and approved prior to delivery to the site. The fill should be placed in lifts with a maximum loose lift thickness of 300 mm and compacted by suitable compaction equipment. Fill placed beneath the building should be compacted to a minimum of 98% of the materials SPMDD.

Site-excavated soil could be placed as general landscaping fill and to build up areas that are to be paved. Workable site-excavated material, free of organics and deleterious materials should be spread in maximum 300 mm thick loose lifts and compacted by several passes of a suitably sized sheepsfoot vibratory roller and reviewed by Paterson personnel at the time of construction. It is recommended that site-generated fill that may be used for raising the subgrade for settlement sensitive areas be reviewed and approved by Paterson personnel prior to re-use.

The fill should be prepared by segregating all cobbles and boulders larger than 200 mm in diameter, significant amounts of organics (i.e., peat, topsoil, roots, stumps, logs, etc.) and inorganic debris (i.e., construction debris, plastics, PVC, metals, etc.). Sampling and testing of the fill material for grain-size distribution and standard proctor values should be completed by Paterson prior to re-use of the subject fill. Frozen material may not be considered for this purpose. This process should be reviewed and approved by Paterson field personnel upon completion of each lift.

Protection of Subgrade for Raft Foundation – Southern Parcel

Where a raft foundation is utilized, it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed, silty clay subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance to the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the silty clay to potential disturbance due to drying, and immediately (i.e., within 48 hours) of exposing the clay bearing medium. It should be understood that the mud slab alone is not considered sufficient to mitigate the potential for the migration of frost within the clay bearing medium if construction is undertaken during winter conditions.

Compacted Granular Fill Working Platform (Deep Foundation)

Should the proposed building(s) be supported on a deep foundation, the use of heavy equipment would be required to install the piles. It is conventional practice to install a compacted granular fill layer, at a convenient elevation, to allow the equipment to access the site without getting stuck and causing significant disturbance. A typical working platform could consist of 600 mm of OPSS Granular B Type II crushed stone which is placed and compacted to a minimum of 98% of its SPMDD in lifts not exceeding 300 mm in thickness.

Once the piles have been installed and cut off, the working platform can be regraded, and soil tracked in, or soil pumping up from the pile installation locations can be bladed off and the surface can be topped up, if necessary, and recompacted to act as the substrate for further fill placement for the floor slab.

5.3 Foundation Design

Bearing Resistance Values – Conventional Spread Footings

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

Engineered fill placed for the purpose of overlying previously reviewed and approved proof-rolled fill surface should be placed in accordance with our recommendations in Subsection 5.2 of this report. In summary, the fill should be placed in maximum 300 mm thick loose lifts, compacted to a minimum of 98% of the materials SPMDD and reviewed and approved by Paterson field personnel at the time of construction.

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

Northern Parcel – Building 1

Footings placed on an undisturbed soil bearing surface can be designed using the following bearing resistance values provided in Table 5 below:

Table 5 - Bearing Resistance Values – Northern Parcel

Bearing Surface	Bearing Resistance Values (kPa)	
	SLS	ULS
Approved Proof-Rolled Fill	175	260
Very Stiff to Stiff Brown Silty Clay	175	260

Note: Strip and pad footings can be designed using the bearing resistance values provided for an approved proof-rolled fill bearing surface, provided that fill placement is completed in accordance with Subsection 5.2 of this report.

Southern Parcel – Building 14

Footings placed on an undisturbed soil bearing surface can be designed using the following bearing resistance values provided in Table 6 below:

Table 6 - Bearing Resistance Values – Southern Parcel

Bearing Surface	Bearing Resistance Values (kPa)	
	SLS	ULS
Very Stiff to Stiff Brown Silty Clay	175	260
Stiff to Firm Grey Silty Clay	75	110

Note: Strip footings up to 1.5 m wide and pad footings up to 3 m wide can be designed using the bearing resistance values provided for an undisturbed, brown silty clay bearing surface at or above geodetic elevation 85.5m.

Strip and pad footings, up to 3 m wide can be designed using the bearing resistance values provided for an undisturbed, grey silty clay bearing surface below geodetic elevation 85.5m.

Raft Foundation – Southern Parcel – Building 14

For support of the proposed development, consideration could be given to using a raft foundation if design building loads exceed the design bearing resistance values provided for conventional spread footings. It is understood that the proposed building will not be provided with basement levels.

It should be noted that at the time of writing this report, information related to the underside of raft elevation as well as raft thickness was not available. For design purposes, it was assumed that the base of the raft foundation would be located at a geodetic elevation of 86.5 m.

It should be noted that the currently provided raft slab bearing resistance value is considered subject to further review based on the actual potential excavation depth and founding elevation. Therefore, the current design value is considered preliminary and subject to further review and coordination during the future design phase. It is anticipated the raft will be founded below the depth of frost migration for heated structures, or a minimum of 1.5 m below the finished grade surround the structure.

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.

For the raft slab foundation, a bearing resistance value at SLS (contact pressure) of **70 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff brown silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **105 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on the understanding that the proposed building will not be provided with a basement level and be of slab-on-grade construction, it is expected that the raft foundation will be installed on the brown silty clay deposit. The modulus of subgrade reaction was calculated to be **2.8 MPa/m** for a contact pressure of **105 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Based on the following assumptions for the raft foundation, the proposed buildings can be designed using the above parameters with a total and differential settlement of 25 and 19 mm, respectively.

Deep Foundation – End Bearing Piles

It is understood that a deep foundation, consisting of end bearing piles, will be considered for the foundation support of the proposed building. For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 7. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

Table 7 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	925	1,100	9	27
245	11	1,050	1,250	9	31
245	13	1,200	1,400	9	35

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving.

A full-time field review program should be conducted by Paterson field personnel during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical). Paterson may undertake dynamic pile testing at the time of pile driving and planning. The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

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

Foundation Uplift Resistance

Uplift forces on the proposed foundations can be resisted using the dead weight of the concrete foundations, the weight of the materials overlying the foundations, and the submerged weight of the piles.

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

A sieve analysis and standard Proctor test should be completed on each of the fill materials proposed to obtain an accurate soil density to be expected, so the applicable unit weights can be estimated. Unit weights of materials are provided in Table 8.

Table 8 - Geotechnical Parameters for Uplift and Lateral Resistance Design

Material Description	Unit Weight (kN/m^3)		Internal Friction Angle () φ'	Friction Factor, tan δ	Earth Pressure Coefficients		
	Drained γ_{dr}	Effective γ'			Active K_A	At-Rest K_O	Passive K_P
OPSS Granular A (Crushed Stone)	22.0	13.7	38	0.60	0.22	0.36	8.8
OPSS Granular B, Type II (Well-Graded Sand-Gravel)	21.5	13.4	36	0.55	0.26	0.41	7.5
In Situ Silty Clay	17.0	10.0	33	0.40	0.30	0.45	3.4
Existing Fill Material	18.0	11.0	33	0.5	0.29	0.46	3.39

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

Lateral Support

The bearing medium for foundations is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils 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.

Permissible Grade Raise

Based on the undrained shear strength values of the silty clay deposit encountered throughout the subject site, a permissible grade raise restriction geodetic elevation of **88.0 m** is recommended for areas where building foundations are founded over a silty clay deposit. Areas affected by a permissible grade raise restriction due to the presence of a silty clay deposit are indicated in Drawing PG7572-2 - Permissible Grade Raise Plan in Appendix 2. Footings placed within the northern parcel and bearing on an approved fill bearing surface overlying the in-situ glacial till deposit will not be subjected to permissible grade raise restrictions.

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. In the event that site-generated silty clay is considered for use as grade raise fill, Paterson should be advised as to verify if permissible grade raise restrictions may be reduced.

It should be noted that foundation bearing resistance values are reliant on anticipated grade raise fill. It is imperative that the grade raise restrictions outlined above are not exceeded without approval from Paterson.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Site Designation X_D** for foundations constructed at the subject site in accordance with the 2024 Ontario Building Code (OBC 2024). If a higher seismic site class is required, a site-specific shear wave velocity test may be completed to accurately determine the applicable seismic site classification for foundation design of the proposed buildings.

The soils underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the 2024 Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Slab on Grade Construction

With the removal of all topsoil and fill containing significant amounts of deleterious or organic materials, the native soil or existing fill subgrade reviewed and approved by Paterson field personnel at the time of excavation will be considered an acceptable subgrade surface on which to commence backfilling for slab-on-grade construction.

Where the subgrade consists of existing fill, it is recommended that the fill layer be proof-rolled (i.e., re-compacted) using a suitably sized vibratory sheepsfoot roller completing several passes over the subgrade and be reviewed and approved by Paterson field personnel at the time of construction. This is described further in Subsection 5.2 of this report. Any poor performing areas observed throughout proof-rolling or soft areas should be removed and reinstated with an engineered fill, such as OPSS Granular A or Granular B Type II placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the material's SPMDD.

OPSS Granular A or Granular B Type II are recommended for backfilling and raising the subgrade level below the floor slab. It is recommended that the upper 200 mm sub-floor fill consists of OPSS Granular A crushed stone. All backfill material within the footprint of the proposed buildings should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

All grade raise fill used to raise the subgrade to the underside of the slab-on-grade should be placed in maximum 300 mm thick loose lifts, compacted to a minimum of 98% of the materials SPMDD. Reference should be made to Subsection 5.2 of this report for additional information pertaining to slab-on-grade construction and proof-rolling the in-situ fill material that is anticipated to be encountered below the floor slab footprint.

5.6 Pavement Design

Pavement Structure Design

Car only parking areas and access lanes are proposed as part of the development at this site. The proposed pavement structures are shown in the following tables below.

Table 9 – Recommended Pavement Structure – Car Parking Areas

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

Table 10 – Recommended Pavement Structure – Local Roadways, Bus and Access Lanes

Thickness (mm)	Material Description
40	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Upper Binder Course** – HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ soil, or sand/crushed stone material placed over the in-situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material. Additional information is provided in the following paragraphs with regards to construction traffic and haul roads that may consider the use of the above-noted pavement structures.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable compaction equipment, noting that excessive compaction can result in subgrade softening.

Pavement Joint Tie-in

Where the proposed pavement structure meets an existing pavement structure, such as an existing road, the following recommendations should be followed:

☐ A 300 mm wide section of the existing asphalt roadway should be saw cut from the existing pavement edge to provide a sound surface to abut the proposed pavement structure.
☐ It is recommended to mill a 300 mm wide and 40 mm deep section of the existing asphalt at the saw cut edge.
☐ The proposed pavement structure subbase materials should be tapered no greater than 3H:1V to meet the existing subbase materials.
☐ Clean existing granular road subbase materials can be reused upon assessment by Paterson at the time of excavation (construction) as to its suitability.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

Since the buildings will consist of a slab-on-grade, a perimeter foundation drainage system is considered optional throughout the landscaped portions of the proposed building footprint.

In areas where hard-scaping or pavement structures will abut the building footprint, it is recommended to implement a foundation drainage system or to backfill the perimeter foundation wall sub-excavation with non-frost susceptible, free-draining crushed stone or clean sand fill as described in the following section. The system should consist of a 100 to 150 mm diameter perforated corrugated plastic pipe wrapped in a geosock and surrounded by 150 mm of 10 mm clear crushed stone. The clear stone should be wrapped in a non-woven geotextile. The foundation wall should be fitted with a a drainage geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx extending to the perimeter drainage pipe with the geotextile face of the composite drainage board facing the backfill. The pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Provided the backfill material consists of one of the previously noted imported fill materials, a composite foundation drainage board system would not be required from a geotechnical perspective.

Alternatively, the perimeter drainage pipe may be placed up to 600 mm below proposed finished grade and against the building footprint upon site-generated compacted soil backfill to ensure adequate drainage of the overlying granular fill layer is provided from precipitation events and/or spring meltwater.

In this configuration, provided the backfill overlying the pipe consists of crushed stone fill associated with the pavement structure, a composite foundation drainage board will not be required. The installation of the perimeter drainage system should be reviewed by Paterson personnel at the time of construction.

Foundation Backfill

Backfill against the exterior sides of the foundation walls should consist of free draining non-frost susceptible granular materials (such as clean sand or OPSS Granular B Type I or Type II granular material) or site-generated workable soils placed in maximum 300 mm thick loose lifts and compacted using suitably sized compaction equipment as described in Subsection 5.2 of this report.

The greater part of the site excavated materials will be frost susceptible and, as such, are recommended to be provided drainage as identified in the preceding paragraphs. Consideration could however be given to reusing the crushed stone fill generated from the excavation of Building 1 within the northern parcel as non frost-susceptible, free-draining foundation backfill for Building 14, however this would not apply or be considered suitable for clay soils generated from the Building 1 sub-excavations.

Building entrances and areas where the ground surface is sensitive to heave and settlement should be provided with adequate frost protection to mitigate the backfill from heaving in response to freezing conditions.

Paterson may advise on suitable combinations of insulation and backfill during the design stage, and prior to tendering, for these areas once detailed grading information is known and able to be reviewed in detail from a geotechnical perspective.

Concrete Sidewalks Adjacent to Buildings

It is recommended all hardscaping (i.e., sidewalks and concrete pathways) surrounding the buildings be underlain by a minimum of 150 mm of OPSS Granular A and further by 450 mm of OPSS Granular B Type II crushed stone placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the materials SPMDD. The engineered fill layers should be placed on appropriately prepared subgrade and foundation wall backfill as identified herein and in Section 5.2 of this report.

The longevity of hardscaping will be improved if founded upon site-generated soil backfill and provided with subgrade drainage by the use of the subdrain system identified herein. Additional drainage is not considered beneficial if the foundation backfill layer would consist of imported crushed stone fill for the height of the foundation and footprint of the hardscaping.

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. A minimum 1.5 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation, should be provided for adequate frost protection of heated structures.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action. These should be provided with a minimum 2.1 m thick soil cover, or an equivalent thickness of soil cover and foundation insulation.

6.3 Excavation Side Slopes

The side slopes of the excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is expected that sufficient room will be available for the greater part of the excavation to be undertake by open-cut methods (i.e., unsupported excavations), with the exception of the where the proposed structures are in close proximity to their associated property lines and adjacent roadways/structures.

Unsupported Excavations

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

Excavation side slopes for the building footprint are recommended to be provided surface protection from erosion by rain and surface water runoff if shoring is not anticipated to be implemented. This can be accomplished by covering the entire surface of the excavation side-slopes with tarps secured between the top and bottom of the excavation and approved by Paterson personnel at the time of construction. It is further recommended to maintain a relatively dry surface along the bottom of the excavation footprint to mitigate the potential for sloughing of side-slopes. All efforts should be made to maintain dry excavation areas for servicing and associated short-term temporary excavation works.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides. It is recommended that a trench box be used at all times to protect personnel working in trenches. Based on this, trench boxes should be considered for all sewer pipe installations undertaken throughout the subject site. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

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

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 system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures, and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored, or braced.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur, and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The tie-back anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended. Further, the bonded portion of the rock anchor should be fully extended below the sound bedrock surface and should located completely below the weathered portion of the bedrock formation.

It is recommended that the anchor drill hole diameter be within 1.5 to 2 times the rock anchor tendon diameter and the anchor drill holes be inspected by geotechnical personnel and should be flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

The earth pressures acting on the shoring system may be calculated using the parameters provided in Table 11.

Table 11 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	18
Submerged Unit Weight (γ'), kN/m^3	13

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

The 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 should be calculated full weight, with no hydrostatic groundwater pressure component.

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

6.4 Pipe Bedding and Backfill

The pipe bedding for the sewer and water pipes should consist of at least 150 mm of OPSS Granular A. The bedding layer thickness should be increased to a minimum of 300 mm where the subgrade will consist of grey silty clay. The material should be placed in a maximum 225 mm thick loose lifts and compacted to a minimum of 95% of its SPMDD.

The bedding material should extend at least to the spring line of the pipe. The cover material, which should consist of OPSS Granular A, should extend from the spring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 225 mm thick lifts and compacted to a minimum of 95% of its SPMDD.

Further, trench excavations advanced below the clay deposit and throughout the upper portion of the stiff clayey glacial till may require bedding thickness in the range of 500 to 600 mm and wrapped in geogrid to consider the lesser stiff nature of the in-situ soils at those depths. It is recommended that Paterson review and advise on the necessity for this consideration once detailed service drawings are available for further review.

Reinstatement of the trench located above the pipe cover layer should consist of placing trench-generated workable soil fill (i.e., grey clay is not expected to be workable or suitable for this purpose) in maximum 300 mm thick loose lifts and compacted using a suitably sized vibratory sheepsfoot roller to a minimum of 95% of the materials SPMDD, or, making several passes (i.e., a minimum of 5 to 6) under the supervision of Paterson field personnel.

Lateral support zones for pipes may be considered as 1.5H:1V for sewers place in overburden and should be considered during the site sewer design to mitigate potential undermining of shallower sewer pipes by adjacent, deeper sewers.

Each lift of soil fill placed within the service trenches should be reviewed and approved at the time of construction by Paterson personnel. Wet site-generated fill, such as the grey silty clay, will be difficult to re-use, as the high-water contents make compacting and using the backfill impractical without an extensive drying period.

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

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

Backfilling Within Trench Boxes

When the bedding and cover material is placed within the confines of a trench box and steel plates, it is recommended that the trench box be placed tightly against the outside of the trench walls and remains approximately 300 mm above the obvert level of the service pipe.

The vertical excavation sidewalls within the lower portion of the trench (below the obvert level of the pipe) can be supported using steel plates extended down to the bottom of the trench. The steel plates can be extended below the base of the excavation to prevent basal heave, in conjunction with adequate dewatering measures when located below the groundwater table.

To minimize the potential for disturbance of the bedding and cover material and subsequent settlement of the service pipe during the removal of the steel plates, it is recommended that the bedding layer be re-compacted tightly against the trench sidewalls upon removal/lifting of the steel plate up to the top of the bedding layer and prior to placing the pipe.

This is recommended to mitigate settlement of the pipe that would result from removing the plates without re-compacting the fill that would be left unconfined to the sides of the trench. This procedure would be repeated for the spring line and cover layers until the steel plates are removed. It is generally recommended that this procedure be reviewed by Paterson field personnel at the time of construction.

6.5 Groundwater Control

It is anticipated that groundwater infiltration into the excavations should be low to moderate and controllable using open sumps. All contractors should be prepared to undertake all earthworks in the dry, and to direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium. Due to the presence of lesser permeable materials below the crushed stone fill throughout the northern parcel, it is expected that a notable amount of perched groundwater may be encountered within the building sub-excavations. Therefore, provisions should be carried to manage water levels throughout this area accordingly.

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).

Long-term Groundwater Control

Provided recommendations such as clay seals and landscaping are followed during the design and construction stages, it is not anticipated the proposed development will negatively impact the groundwater table surrounding the area of the subject site and associated structures and infrastructure from a geotechnical perspective.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur. In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures using straw, propane heaters and tarpaulins or other suitable means.

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required. The trench excavations should be carried out in a manner that will avoid the introduction of frozen materials into the trenches. Also, pavement construction is difficult during winter.

The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. In addition, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure. Additional information could be provided, if required. Provisions should also be carried out for accommodating spring-thaw conditions when subgrade conditions for pavements and other works are impacted by higher degrees of soil saturation. Additional information should be provided by Paterson for planning winter construction and pavement works.

6.7 Corrosion Potential and Sulphate

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

6.8 Landscaping Considerations

Tree Planting Restrictions

In accordance with the City of Ottawa Tree Planting in Sensitive Marine Clay Soils (2017 Guidelines), Paterson completed a soils review of the site to determine applicable tree planting setbacks. Atterberg limits testing was completed for the recovered silty clay samples at selected locations throughout the subject site. The results of our testing are presented in Table 1 in Subsection 4.2.

Based on the results of the Atterberg limit testing mentioned above, the plasticity index was found to not exceed 40% for all the tested clay samples. Based on this, the silty clay across the subject site is considered to be a clay of low to medium potential for soil volume change. The following tree planting setbacks are recommended for the low to medium sensitivity silty clay deposit and where trees are located near buildings founded on cohesive soils.

Large trees (mature height over 14 m) can be planted within these areas provided a tree to foundation setback equal to the full mature height of the tree can be provided (e.g. in a park or other green space). Tree planting setback limits **may be reduced to 4.5 m** for small (mature 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. It should be noted that a 1.5 m depth for footings is considered acceptable provided that additional measures be taken. These measures can be discussed upon request under a separate cover.

☐ 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 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 for the side of the buildings facing the subject tree(s) 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), as noted on the subdivision Grading Plan.

It is well documented in literature, and in our experience, that fast-growing trees located near buildings founded on cohesive soils that shrink on drying can result in long-term differential settlements of the structures.

Tree varieties that have the most pronounced effect on foundations are seen to consist of poplars, willows and some maples (i.e. Manitoba Maples) and, as such, they should not be considered in the landscaping design.

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 the preliminary and detailed architectural, structural, grading and servicing plans, from a geotechnical perspective.
- ☐ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.

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

- ☐ Observation of all waterproofing membranes, sub-slab drainage system and all associated systems and assemblies.
- ☐ Observation of all bearing surfaces prior to the placement of concrete.
- ☐ Sampling and testing of the concrete and fill materials.
- ☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ☐ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ☐ Field density tests to determine the level of compaction achieved.
- ☐ Sampling and testing of the bituminous concrete including mix design reviews.

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

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

8.0 Statement of Limitations

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

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

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

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

Paterson Group Inc.

N Versolato

Nicholas F. R. Versolato, CPI, B.Eng.

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
July 4, 2025
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PROVINCE OF ONTARIO

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ATTERBERGS TESTING RESULTS

GRAIN-SIZE TESTING RESULTS

ANALYTICAL TESTING RESULTS

PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371786.91 **NORTHING:** 5026694.14 **ELEVATION:** 87.45

PROJECT: Proposed Residential Development

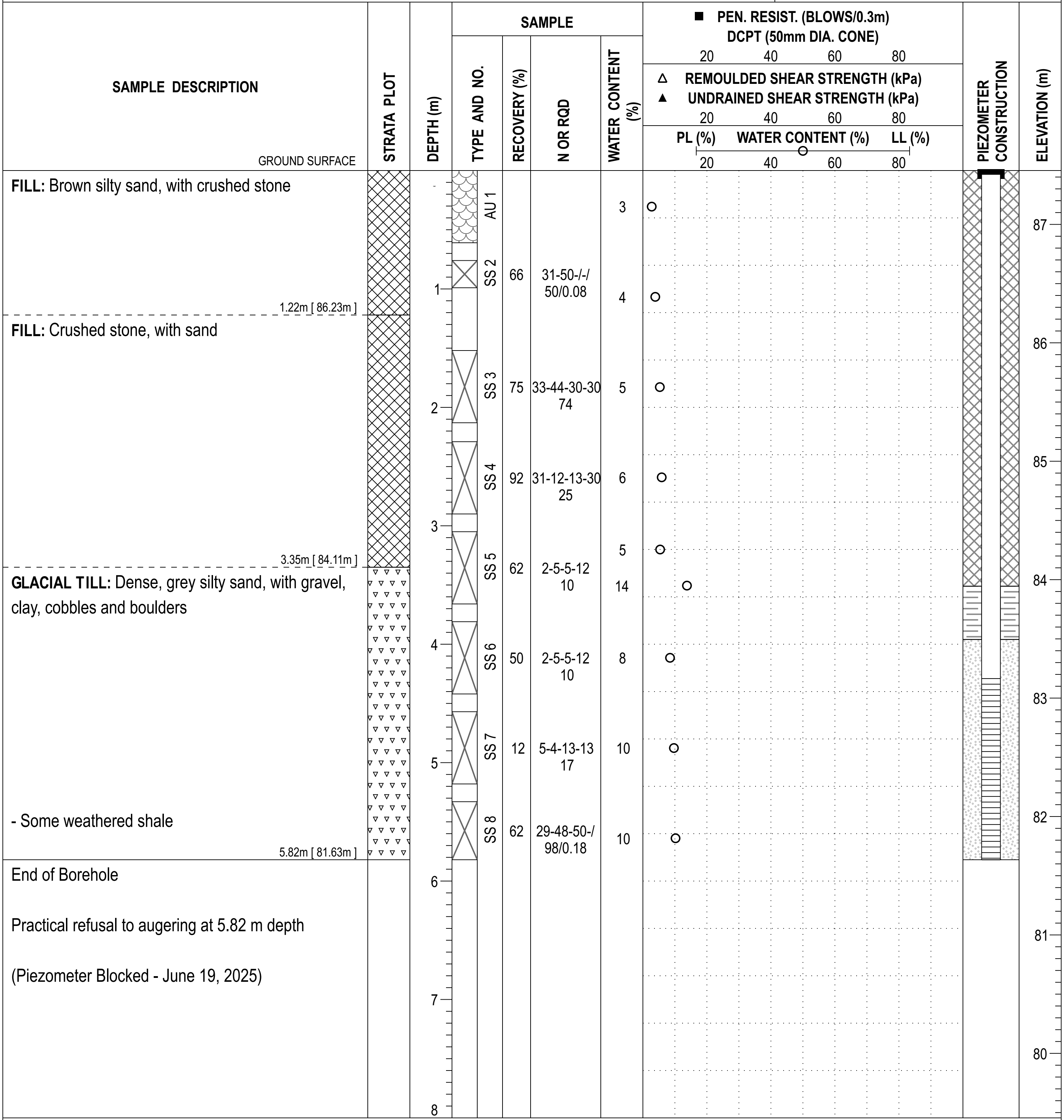
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 5, 2025

FILE NO. : PG7572

HOLE NO. : BH 1-25



SAMPLE DESCRIPTION	
GROUND SURFACE	
FILL: Brown silty sand, with crushed stone	1.22m [86.23m]
FILL: Crushed stone, with sand	3.35m [84.11m]
GLACIAL TILL: Dense, grey silty sand, with gravel, clay, cobbles and boulders - Some weathered shale	5.82m [81.63m]

End of Borehole

Practical refusal to augering at 5.82 m depth

(Piezometer Blocked - June 19, 2025)

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			3
SS 2	66	31-50-/-/ 50/0.08	4
SS 3	75	33-44-30-30 74	5
SS 4	92	31-12-13-30 25	6
SS 5	62	2-5-5-12 10	5 14
SS 6	50	2-5-5-12 10	8
SS 7	12	5-4-13-13 17	10
SS 8	62	29-48-50-/ 98/0.18	10

P:/AutoCAD Drawings/Test Hole Data Files/PG75xx/PG7572/data.sqlite 2025-06-30, 10:19 Paterson_Template MR

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371770.64 **NORTHING:** 5026696.44 **ELEVATION:** 87.65

PROJECT: Proposed Residential Development

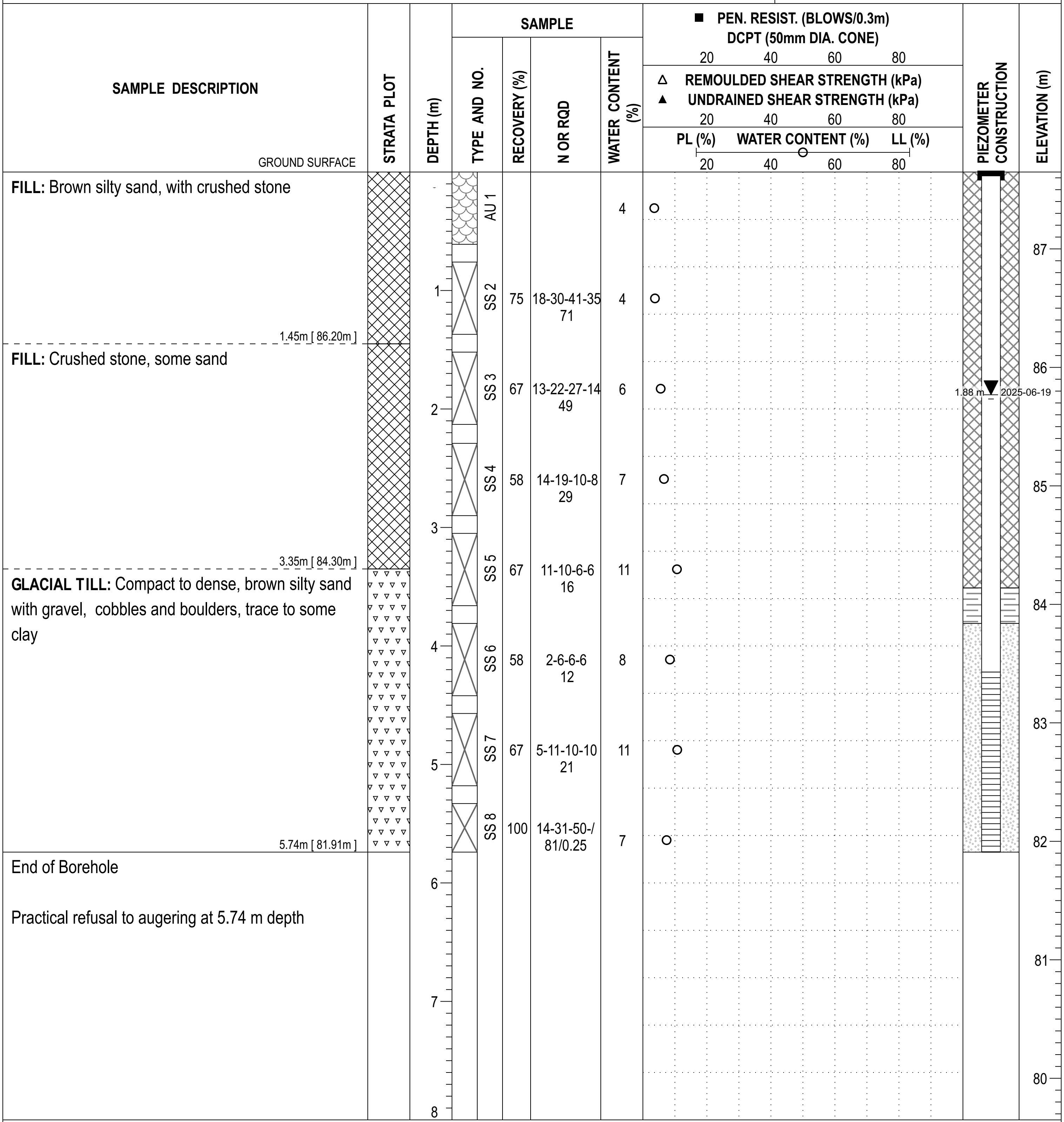
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 5, 2025

FILE NO. : PG7572

HOLE NO. : BH 2-25



FILL: Brown silty sand, with crushed stone

1.45m [86.20m]

FILL: Crushed stone, some sand

3.35m [84.30m]

GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace to some clay

5.74m [81.91m]

End of Borehole

Practical refusal to augering at 5.74 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			4
SS 2	75	18-30-41-35 71	4
SS 3	67	13-22-27-14 49	6
SS 4	58	14-19-10-8 29	7
SS 5	67	11-10-6-6 16	11
SS 6	58	2-6-6-6 12	8
SS 7	67	5-11-10-10 21	11
SS 8	100	14-31-50-/ 81/0.25	7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371763.42 **NORTHING:** 5026677.37 **ELEVATION:** 87.50

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 5, 2025

FILE NO. : PG7572

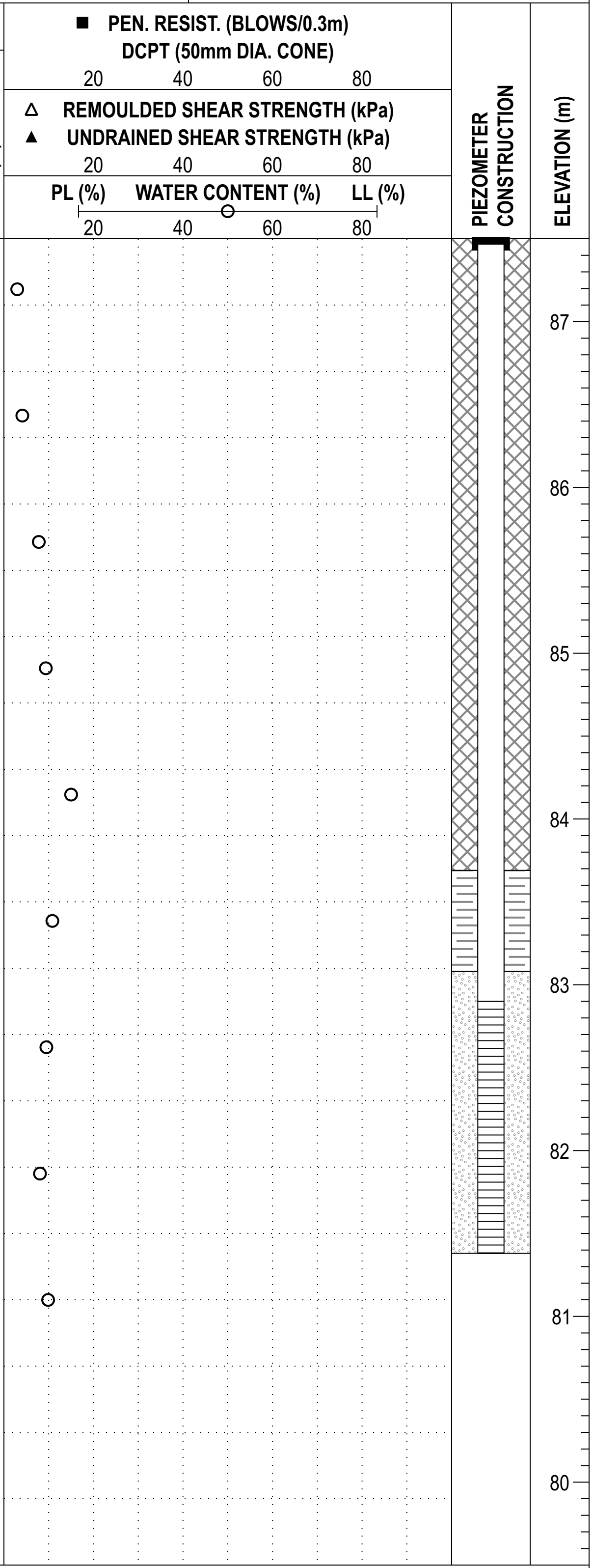
HOLE NO. : BH 3-25

SAMPLE DESCRIPTION	STRATA PLOT
GROUND SURFACE	
FILL: Brown silty sand, with crushed stone 1.45m [86.05m]	▨
FILL: Crushed stone, trace sand 2.97m [84.53m]	▨
GLACIAL TILL: Stiff, brown silty clay, with sand, gravel, cobbles and boulders 5.26m [82.24m]	▽
GLACIAL TILL: Compact, brown silty sand to sandy silt, with gravel, cobbles, boulders and weathered shale 6.12m [81.38m]	▽
End of Borehole Practical refusal to augering at 6.12 m depth (Piezometer Blocked - June 19, 2025)	

	SAMPLE			
	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
	AU 1			3
	SS 2	58	12-19-18-14 37	4
	SS 3	33	4-5-5-5 10	8
	SS 4	29	4-5-5-7 10	9
	SS 5	37	2-4-4-5 8	15
	SS 6	33	3-3-3-3 6	11
	SS 7	29	5-9-10-10 19	9
	SS 8	75	6-8-19-13 27	8
	SS 9	100	50-/-/-/ 50/0.03	
				10

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8

ELEVATION (m): 87, 86, 85, 84, 83, 82, 81, 80



PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371780.71 **NORTHING:** 5026544.05 **ELEVATION:** 86.63

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 5, 2025

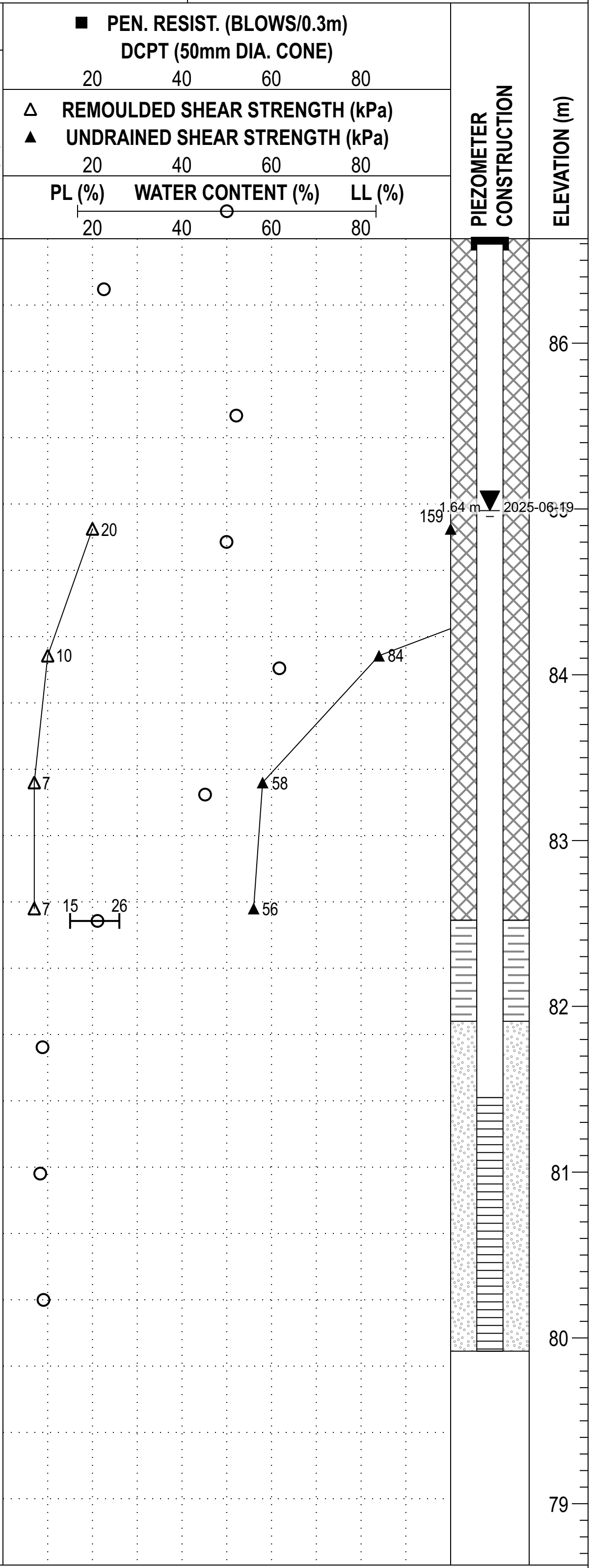
FILE NO. : PG7572

HOLE NO. : BH 4-25

SAMPLE DESCRIPTION	
GROUND SURFACE	
FILL: Brown sandy silt with crushed stone, topsoil and organics	0.69m [85.94m]
Very stiff to stiff, brown **CLAYEY SILT** to **SILTY CLAY**, - Grey and firm by 2.3 m depth	3.73m [82.90m]
GLACIAL TILL: Stiff, grey silty clay, with gravel, cobbles and boulders, some sand	4.50m [82.13m]
GLACIAL TILL: Compact, grey silty sand, with clay, gravel, cobbles and boulders	6.71m [79.92m]
End of Borehole	

SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			23
SS 2	83	2-2-3-3 5	52
SS 3	100	P	50
SS 4	100	P	62
SS 5	100	P	45
SS 6	58	P	21
SS 7	58	2-8-8-8 16	9
SS 8	50	5-6-9-10 15	8
SS 9	62	4-8-8-8 16	9

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8



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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371834.09 **NORTHING:** 5026576.46 **ELEVATION:** 86.65

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

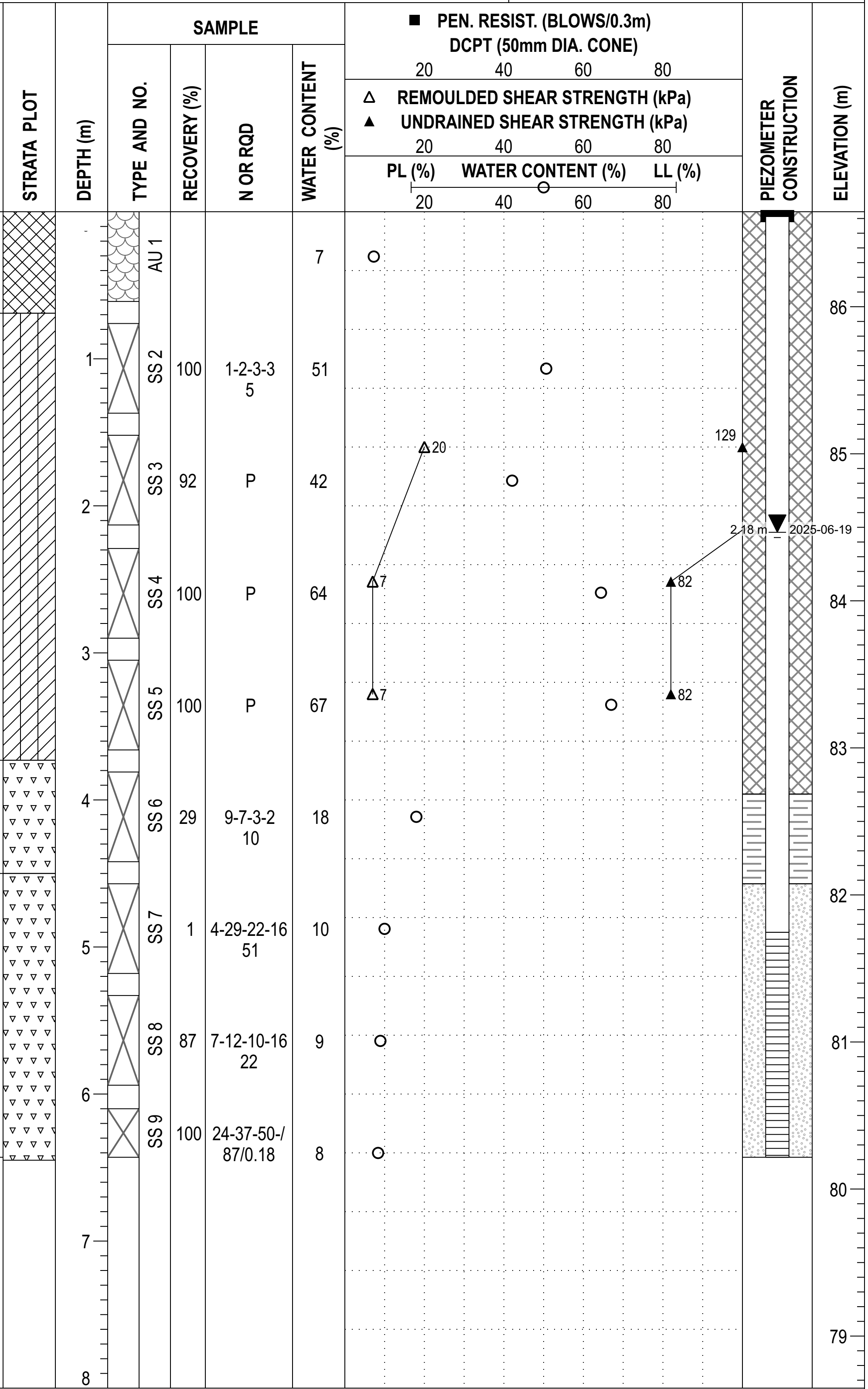
REMARKS:

DATE: June 5, 2025

FILE NO. : PG7572

HOLE NO. : BH 5-25

SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE				
FILL: Brown silty clay, with crushed stone, trace organics	AU 1			7
0.69m [85.96m]				
Very stiff, brown **SILTY CLAY**	SS 2	100	1-2-3-3 5	51
	SS 3	92	P	42
- Grey and stiff by 2.3 m depth	SS 4	100	P	64
	SS 5	100	P	67
3.73m [82.92m]				
GLACIAL TILL: Stiff, grey silty clay, with gravel, cobbles and boulders	SS 6	29	9-7-3-2 10	18
4.50m [82.15m]				
GLACIAL TILL: Dense, grey silty sand, with clay, gravel, cobbles and boulders	SS 7	1	4-29-22-16 51	10
	SS 8	87	7-12-10-16 22	9
	SS 9	100	24-37-50-/ 87/0.18	8
6.43m [80.22m]				
End of Borehole				
Practical refusal to augering at 6.43 m depth				



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PATERSON GROUP

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371829.85 **NORTHING:** 5026588.24 **ELEVATION:** 86.71

PROJECT: Proposed Residential Development

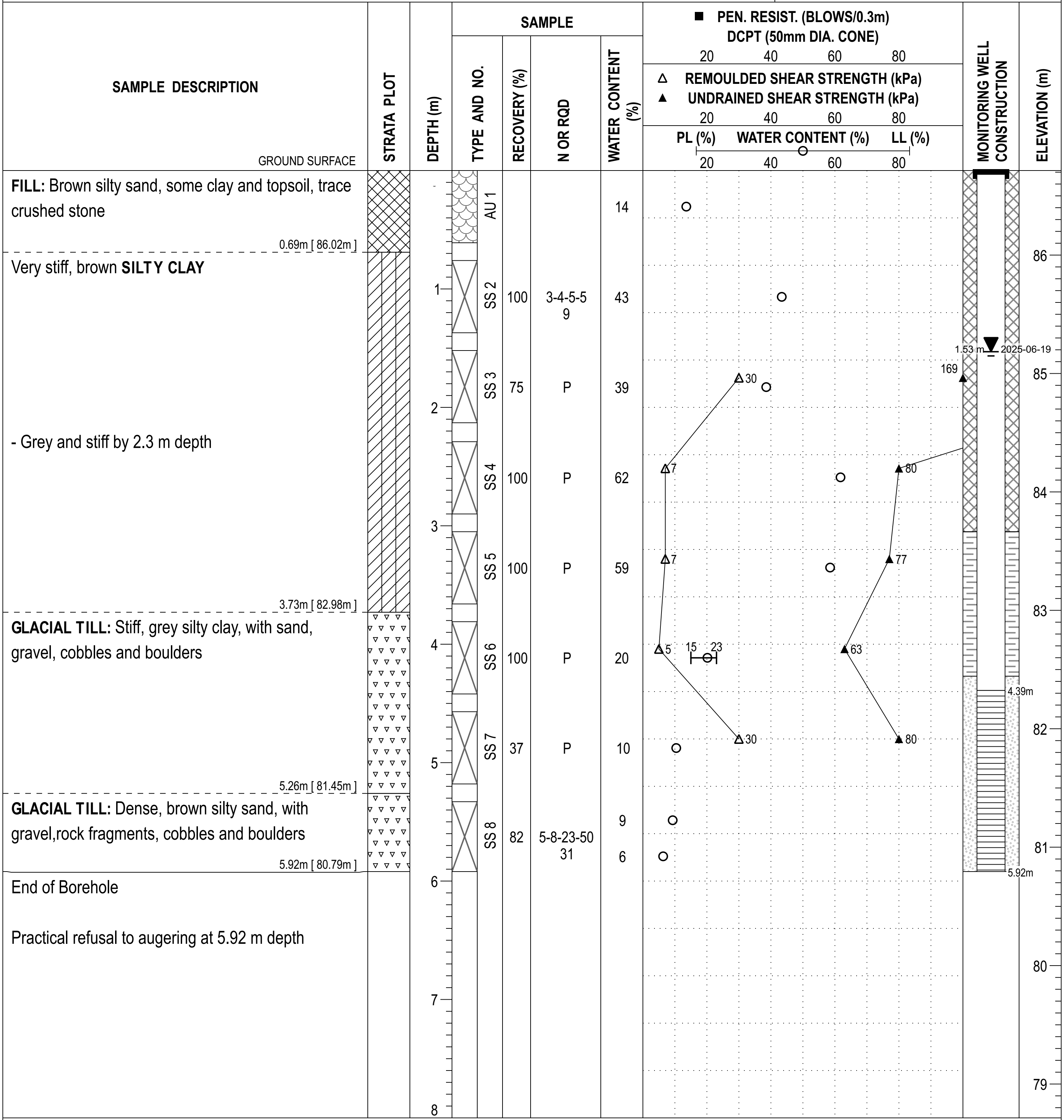
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 6, 2025

FILE NO. : PG7572

HOLE NO. : BH 6-25



SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty sand, some clay and topsoil, trace crushed stone

0.69m [86.02m]

Very stiff, brown **SILTY CLAY**

- Grey and stiff by 2.3 m depth

3.73m [82.98m]

GLACIAL TILL: Stiff, grey silty clay, with sand, gravel, cobbles and boulders

5.26m [81.45m]

GLACIAL TILL: Dense, brown silty sand, with gravel,rock fragments, cobbles and boulders

5.92m [80.79m]

End of Borehole

Practical refusal to augering at 5.92 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			14
SS 2	100	3-4-5-5 9	43
SS 3	75	P	39
SS 4	100	P	62
SS 5	100	P	59
SS 6	100	P	20
SS 7	37	P	10
SS 8	82	5-8-23-50 31	9 6

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1770 Heatherington Road, Ottawa, Ontario

COORD. SYS.: MTM ZONE 9 **EASTING:** 371811.86 **NORTHING:** 5026565.32 **ELEVATION:** 86.66

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 6, 2025

FILE NO. : PG7572

HOLE NO. : BH 7-25

SAMPLE DESCRIPTION

GROUND SURFACE

FILL: Brown silty clay, with crushed stone

0.69m [85.97m]

Stiff, brown **SILTY CLAY**

- Grey and firm by 2.3 m depth

4.50m [82.16m]

GLACIAL TILL: Stiff, grey silty clay, with gravel, cobbles and boulders

5.26m [81.39m]

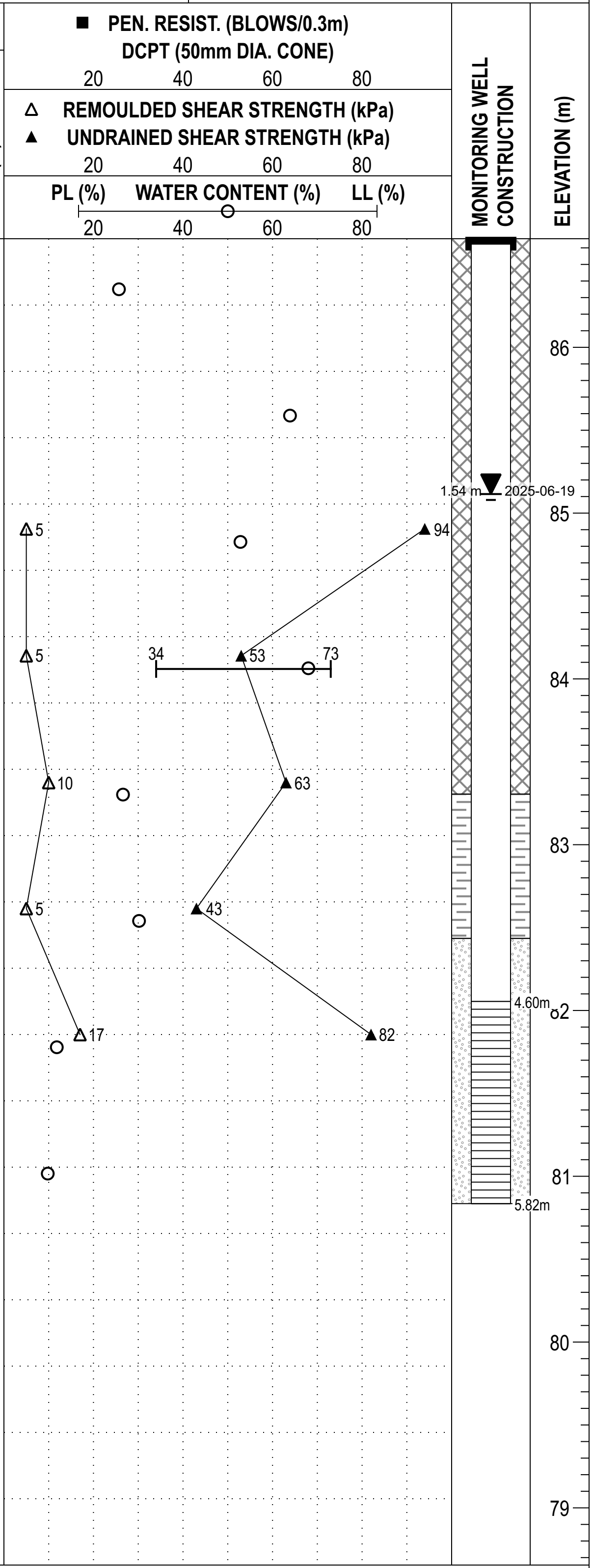
GLACIAL TILL: Dense, brown silt, with gravel, cobbles and boulders

5.82m [80.84m]

End of Borehole

Practical refusal to augering at 5.82 m depth

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			26
SS 2	75	2-2-2-2 4	64
SS 3	58	P	53
SS 4	100	P	68
SS 5	79	P	27
SS 6	54	P	30
SS 7	33	P	12
SS 8	83	9-13-25-50 38	10



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SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

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

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

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

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

CONSOLIDATION TEST

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

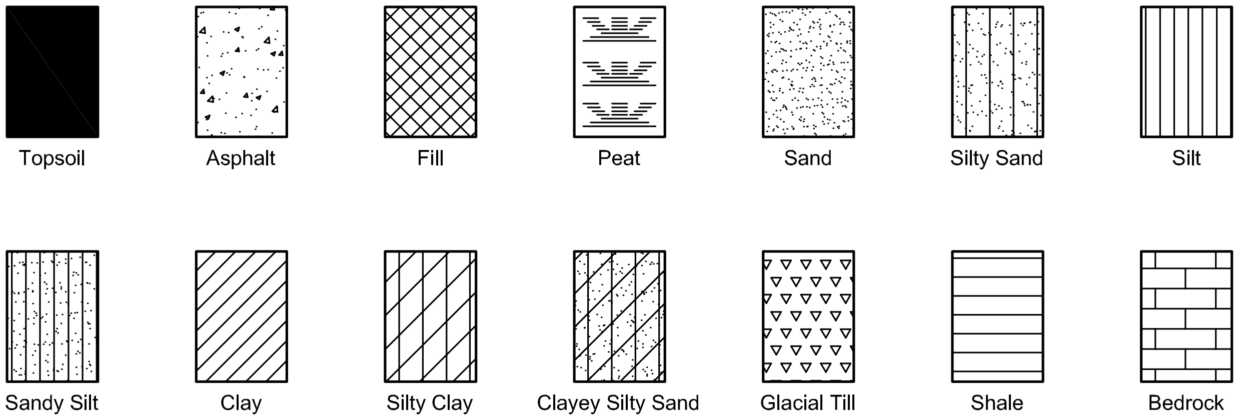
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

PERMEABILITY TEST

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

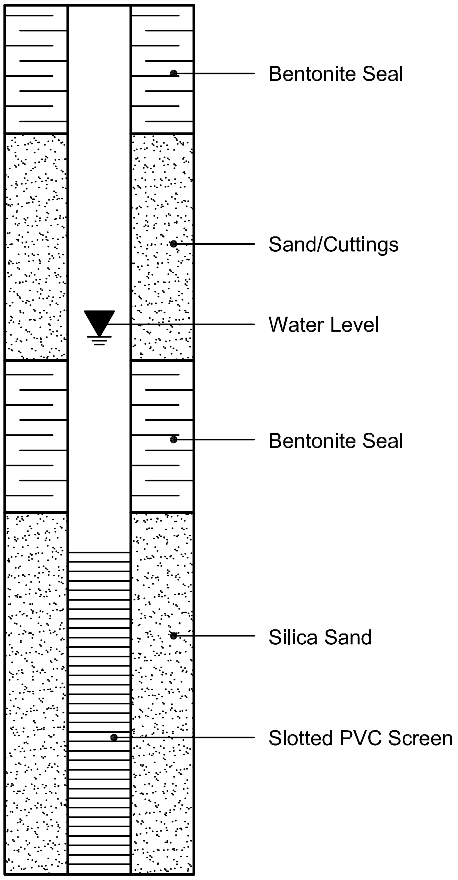
SYMBOLS AND TERMS (continued)

STRATA PLOT

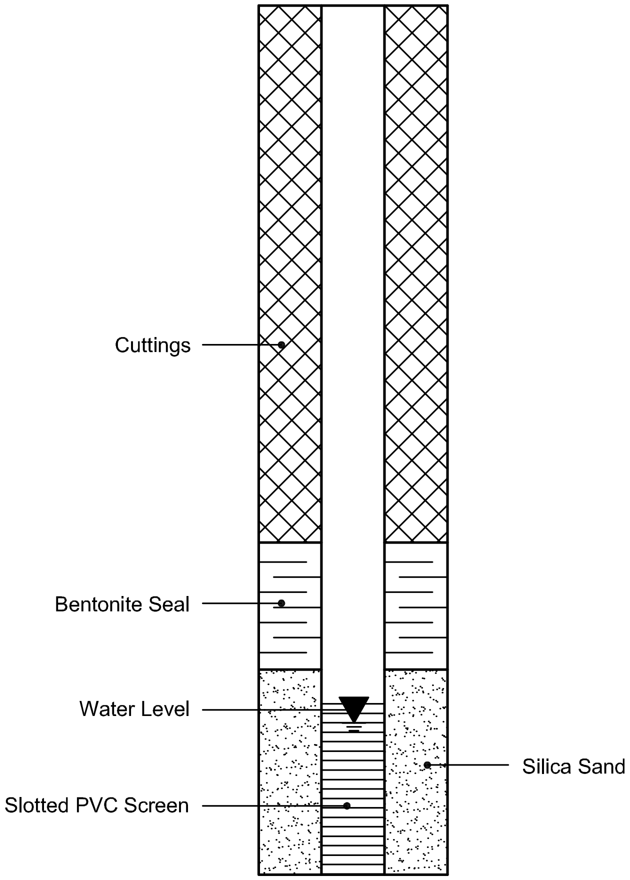


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Log of Borehole BH23-5

Project No: OTT-22026647-A0

Project: Proposed Residential Development

Location: 1770 Heatherington Road, Ottawa, ON

Figure No. 8

Page. 1 of 1

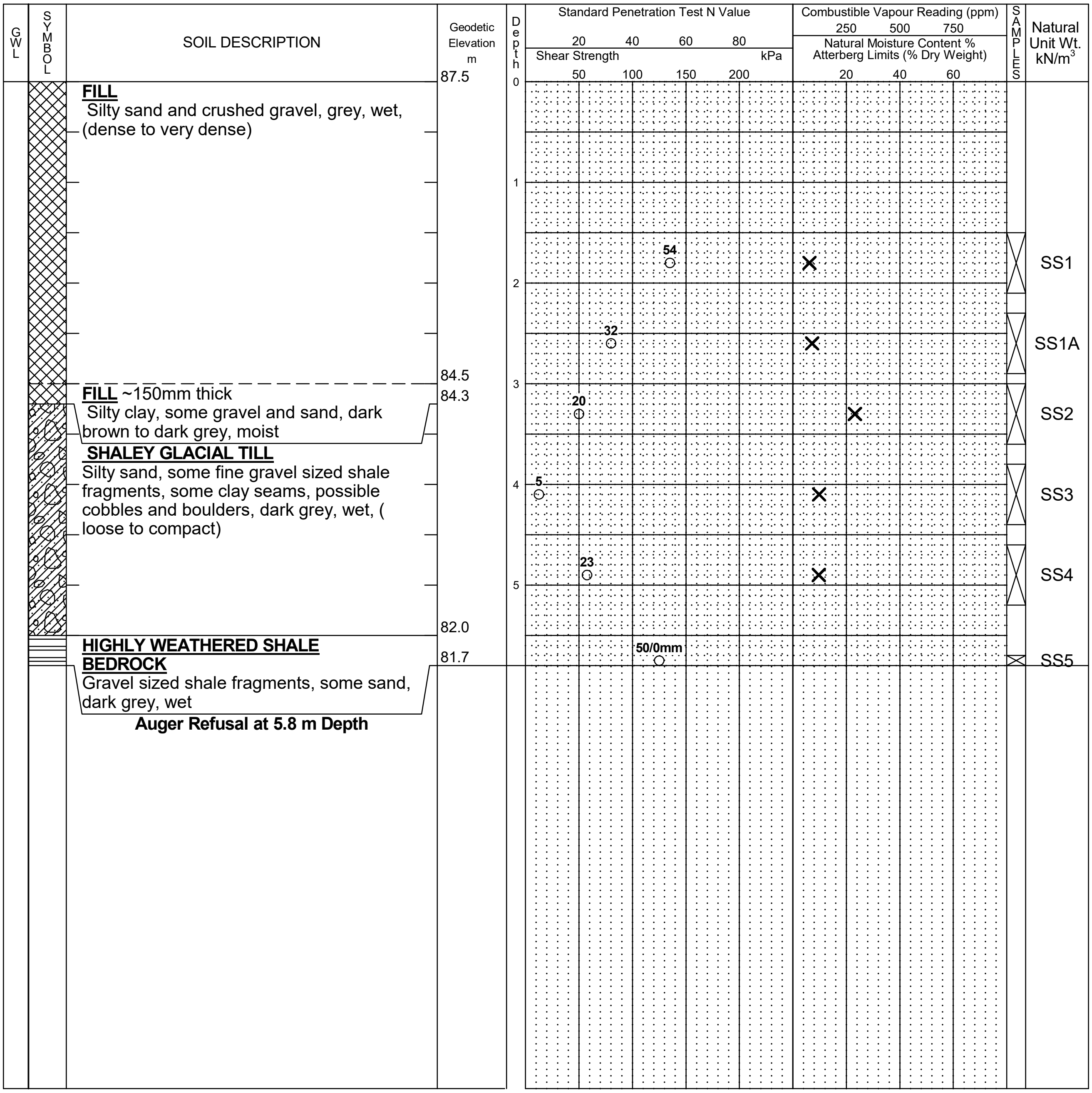
Date Drilled: 'December 12, 2023

Drill Type: CME 45 Track-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.Z. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



Geodetic Elevation m	Soil Description
87.5	**FILL** Silty sand and crushed gravel, grey, wet, (dense to very dense)
84.5	**FILL** ~150mm thick Silty clay, some gravel and sand, dark brown to dark grey, moist
84.3	**SHALEY GLACIAL TILL** Silty sand, some fine gravel sized shale fragments, some clay seams, possible cobbles and boulders, dark grey, wet, (loose to compact)
82.0	**HIGHLY WEATHERED SHALE BEDROCK** Gravel sized shale fragments, some sand, dark grey, wet
81.7	**Auger Refusal at 5.8 m Depth**

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22026647-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
	3.0	5.8

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole BH23-9

Project No: OTT-22026647-A0

Figure No. 12

Page. 1 of 1

Project: Proposed Residential Development

Location: 1770 Heatherington Road, Ottawa, ON

Date Drilled: 'November 21, 2023

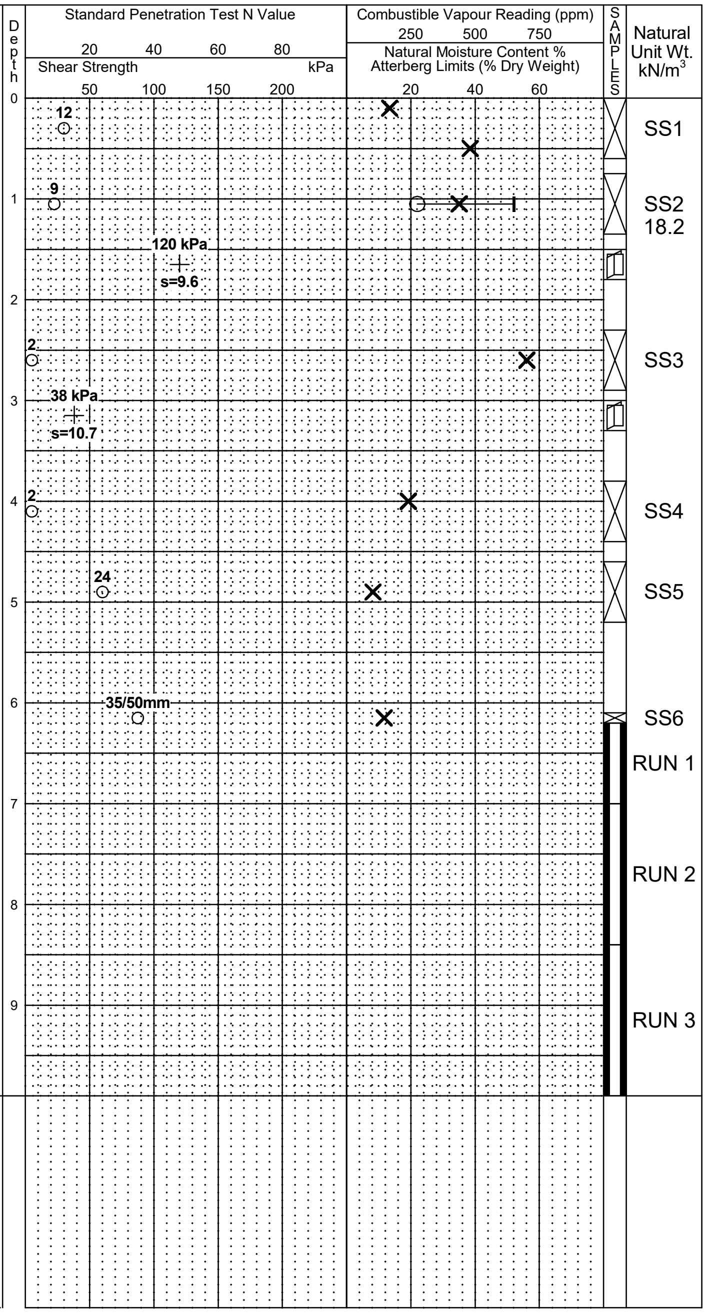
Drill Type: CME 45 Track-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: M.Z. Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	□
Auger Sample	◫	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├─○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m
			86.49
		FILL ~200 mm thick Silty sand, some gravel, brown, moist	86.2
		ORGANIC CLAYEY SILT ~ 100 mm thick Dark brown, moist	86.1
		SILTY CLAY High plasticity, some sand, brown, moist, (very stiff)	
		SILTY CLAY Grey, moist, (firm)	84.4
		SHALEY GLACIAL TILL Silty sand, gravelly (fine shale fragments), trace clay seams, possible cobbles and boulders, moist to wet, (very loose to compact)	82.7
		SHALE BEDROCK Dark grey to black, (poor to fair quality)	80.3
			76.6
		Borehole Terminated at 9.9 m Depth	



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22026647-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	6.2 - 7	100	33
2	7 - 8.4	100	48
3	8.4 - 9.9	100	70

Log of Borehole BH24-14

Project No: OTT-22026647-A0

Project: Proposed Residential Development

Location: 1770 Heatherington Road, Ottawa, ON

Figure No. 16

Page. 1 of 1

Date Drilled: 'March 24, 2024

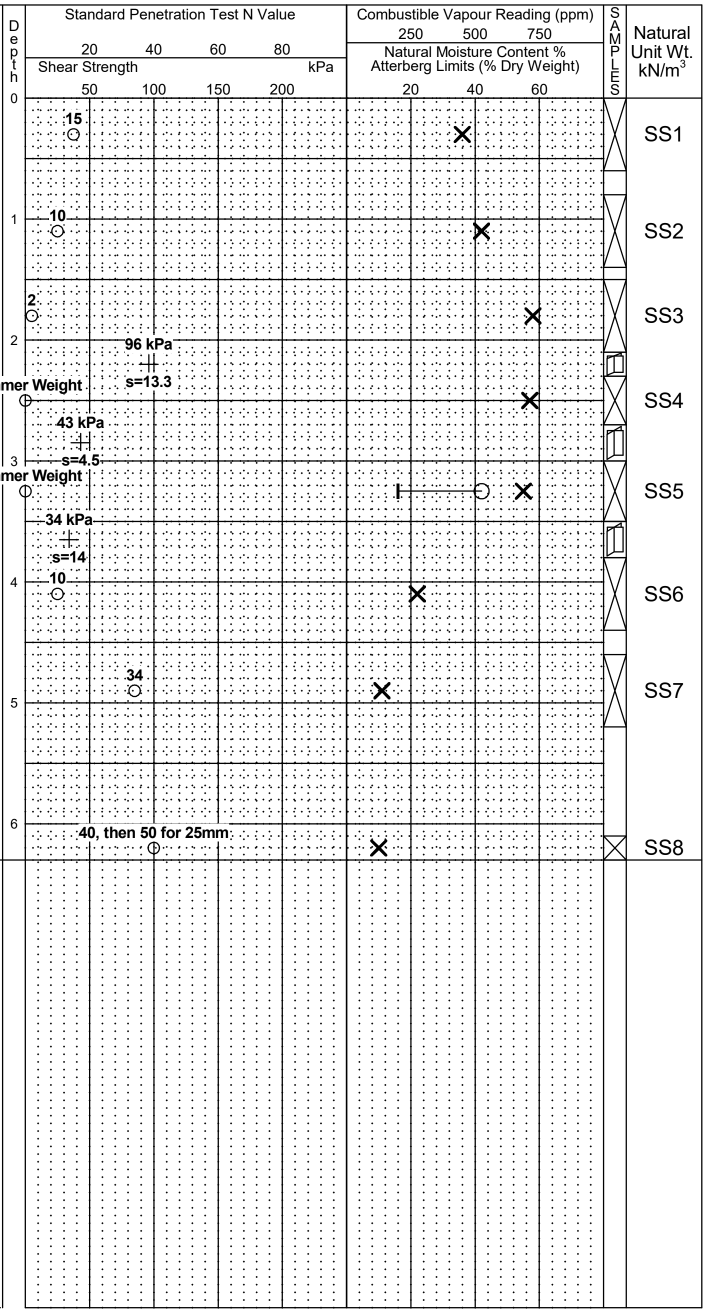
Drill Type: CME 45 Track-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: A.N Checked by: I.T.

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◧	Natural Moisture Content	✕
SPT (N) Value	○	Atterberg Limits	⊢○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m
			86.69
		FILL Mixture of silty sand and silty clay, some topsoil inclusions, dark brown to dark grey, moist, (compact)	
			85.8
		SILTY CLAY Brown, moist, (stiff)	
			84.5
		SILTY CLAY Silty clay of meduim plasticity, trace sand, grey, wet, (firm)	
			82.7
		SHALEY GLACIAL TILL Silty sand, some fine gravel sized shale fragments and clay seams, possible cobbles and boulders, dark grey, wet, (compact to dense)	
			80.4
		Casing Refusal at 6.3 m Depth	



NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole was backfilled upon completion.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-22026647-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole 08-10

Trow

Project No: OTGE00018293JB

Project: Preliminary Geotechnical Investigation

Location: 1770 Heatherington Road, Ottawa, Ontario

Figure No. 3

Feuille. 1 of 1

Date Drilled: 'August 5, 2008

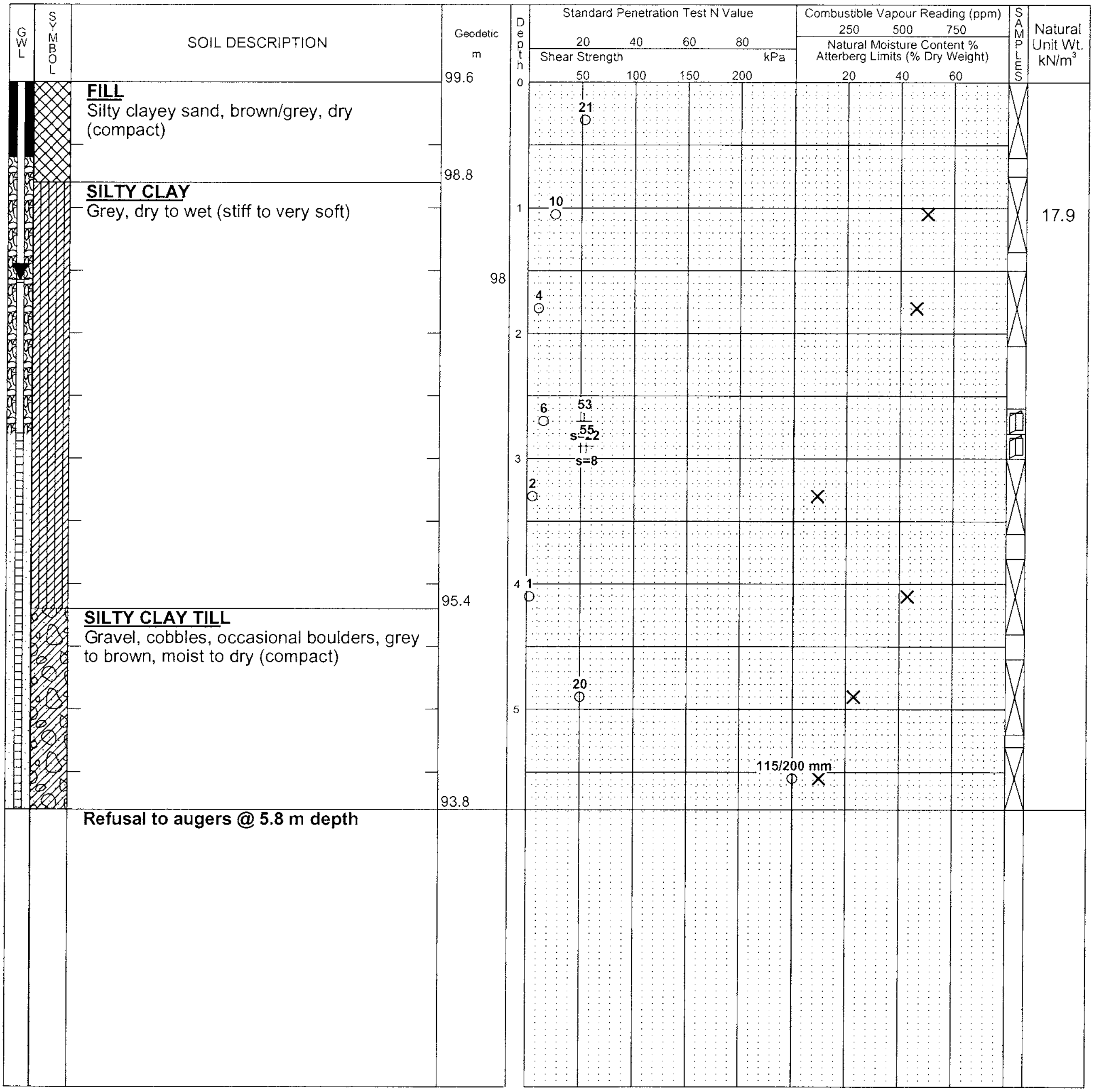
Drill Type:

Datum: Geodetic

Logged by:

Checked by:

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	──	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



FILL
Silty clayey sand, brown/grey, dry (compact)

SILTY CLAY
Grey, dry to wet (stiff to very soft)

SILTY CLAY TILL
Gravel, cobbles, occasional boulders, grey to brown, moist to dry (compact)

Refusal to augers @ 5.8 m depth

NOTES:
1. Borehole/Test Pit data requires Interpretation by Trow before use by others
2. A 19 mm slotted standpipe installed in the borehole upon completion of drilling
3. Field work supervised by a Trow representative
4. See Notes on Sample Descriptions
5. This Figure is to read with Trow Associates Inc. report OTGE00018293JB

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
6 days	1.6	-

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW08-14

Trow

Project No: OTGE00018293JB

Project: Preliminary Geotechnical Investigation

Location: 1770 Heatherington Road, Ottawa, Ontario

Figure No. 7

Feuille. 1 of 1

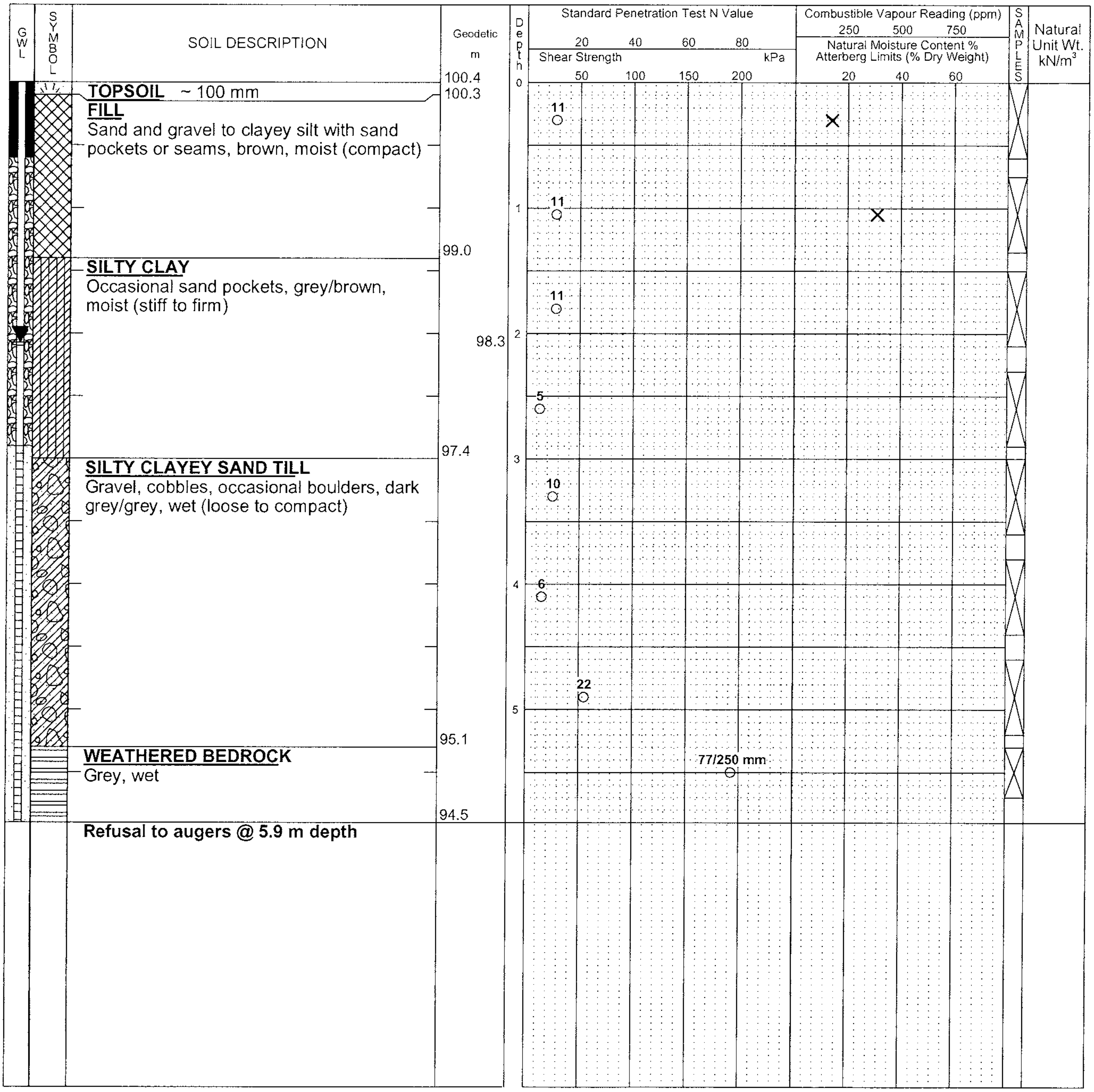
Date Drilled: 'August 6, 2008

Drill Type:

Datum: Geodetic

Logged by: Checked by:

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◫	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	├──○
Dynamic Cone Test	───	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■	Shear Strength by Penetrometer Test	▲
Shear Strength by Vane Test	+ S		



TOPSOIL ~ 100 mm

FILL
Sand and gravel to clayey silt with sand pockets or seams, brown, moist (compact)

SILTY CLAY
Occasional sand pockets, grey/brown, moist (stiff to firm)

SILTY CLAYEY SAND TILL
Gravel, cobbles, occasional boulders, dark grey/grey, wet (loose to compact)

WEATHERED BEDROCK
Grey, wet

Refusal to augers @ 5.9 m depth

NOTES:
1. Borehole/Test Pit data requires interpretation by Trow before use by others
2. Monitoring well installed in the borehole upon completion of drilling
3. Field work supervised by a Trow representative
4. See Notes on Sample Descriptions
5. This Figure is to read with Trow Associates Inc. report OTGE00018293JB

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
5 days	2.1	-

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

Log of Borehole MW08-17

Project No: OTGE00018293JB

Project: Preliminary Geotechnical Investigation

Location: 1770 Heatherington Road, Ottawa, Ontario

Figure No. 10

Feuille. 1 of 1

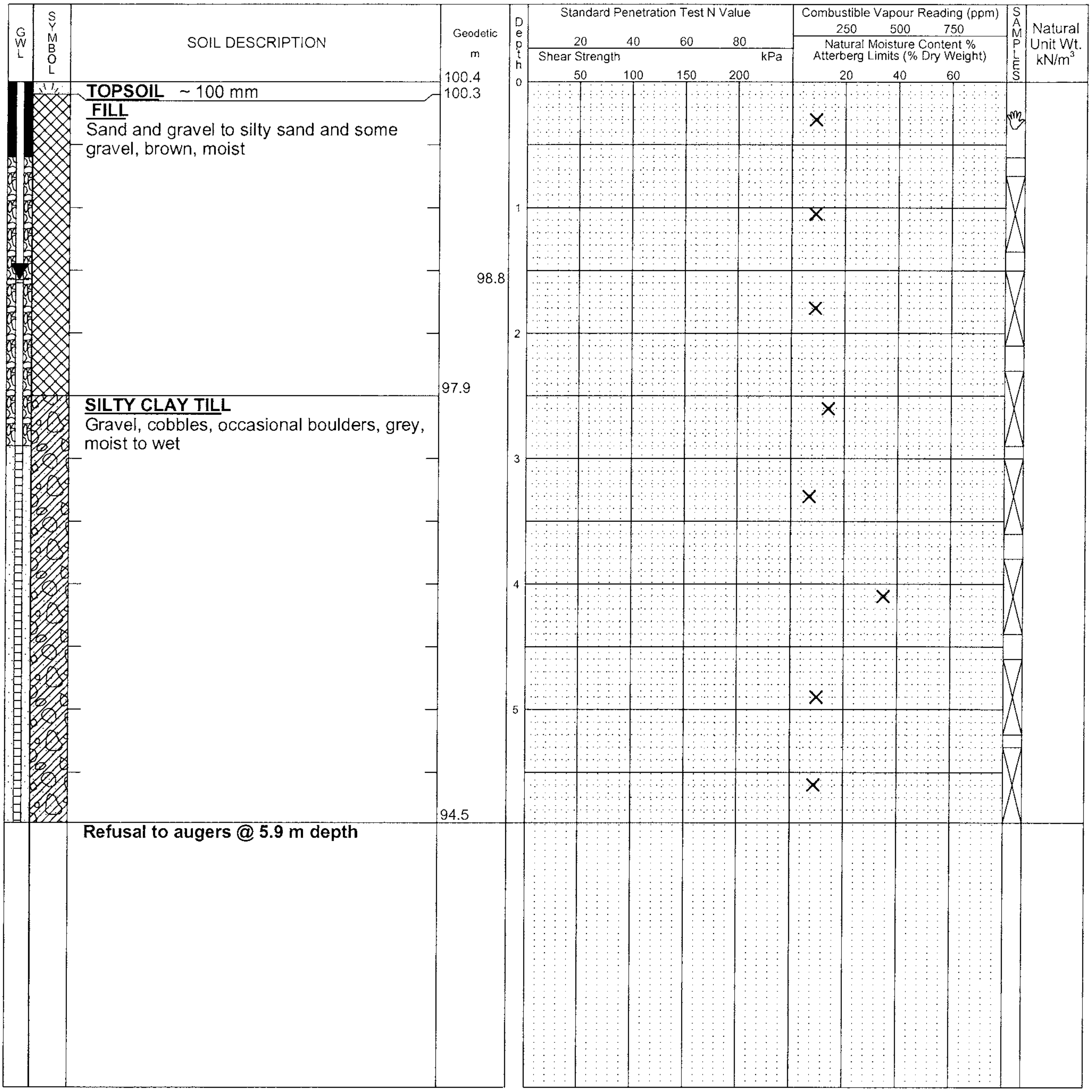
Date Drilled: 'August 6, 2008

Drill Type:

Datum: Geodetic

Logged by: Checked by:

Split Spoon Sample	☒	Combustible Vapour Reading	☐
Auger Sample	◨	Natural Moisture Content	×
SPT (N) Value	○	Atterberg Limits	⊢—○
Dynamic Cone Test	—	Undrained Triaxial at % Strain at Failure	⊕
Shelby Tube	■		
Shear Strength by Vane Test	+ S	Shear Strength by Penetrometer Test	▲



TOPSOIL ~ 100 mm

FILL
Sand and gravel to silty sand and some gravel, brown, moist

SILTY CLAY TILL
Gravel, cobbles, occasional boulders, grey, moist to wet

Refusal to augers @ 5.9 m depth

NOTES:
1. Borehole/Test Pit data requires interpretation by Trow before use by others
2. Monitoring well installed in the borehole upon completion of drilling
3. Field work supervised by a Trow representative
4. See Notes on Sample Descriptions
5. This Figure is to read with Trow Associates Inc. report OTGE00018293JB

WATER LEVEL RECORDS

Elapsed Time	Water Level (m)	Hole Open To (m)
6 days	1.6	-

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %

PATERSON GROUP

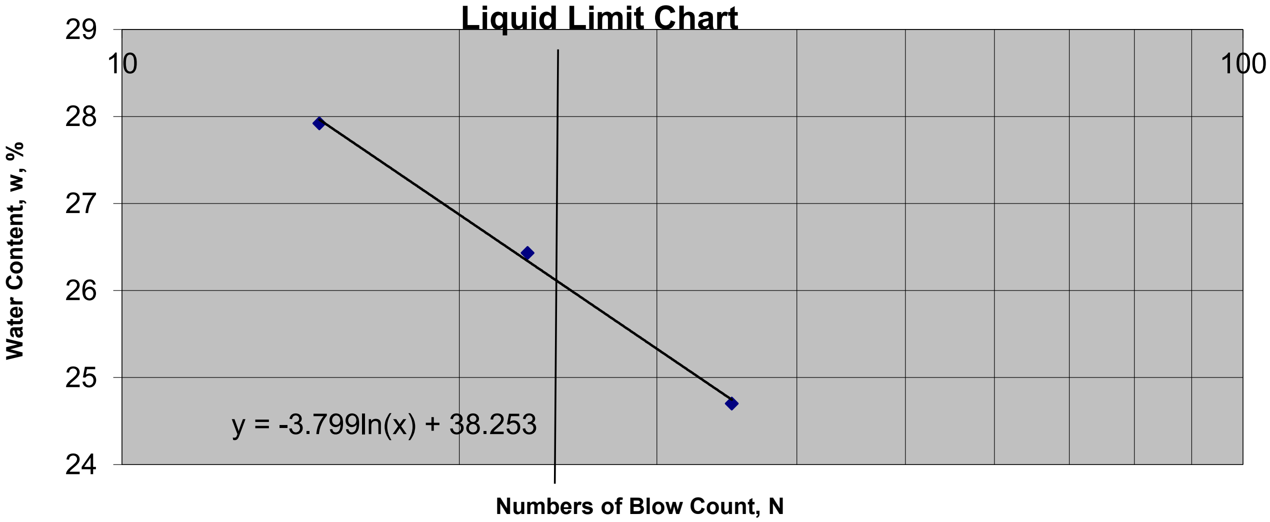
ATTERBERG LIMITS
LS-703/704

CLIENT:	OCH	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	DATE SAMPLED:	06-Jun
LOCATION:	BH4-25 SS6 @ 12'6" - 14'6"	DATE REPORTED:	19-Jun

CAN NO.	**11**	**67**	**32**				
WT. OF CAN	8.64	7.20	4.37				
WT. OF SOIL & CAN	17.3	13.85	11.54				
WT. OF DRY SOIL & CAN	15.41	12.46	10.12				
WT. OF MOISTURE	1.89	1.39	1.42				
WT. OF DRY SOIL & CAN	6.77	5.26	5.75				
WATER CONTENT, w, %	**27.92**	**26.43**	**24.7**				
NO. OF BLOWS, N	15	23	35				

CAN NO.	**12**	**18**
WT. OF CAN	16.73	20.02
WT. OF SOIL & CAN	25.52	28.48
WT. OF DRY SOIL & CAN	24.32	27.37
WT. OF MOISTURE	1.2	1.11
WT. OF DRY SOIL & CAN	7.59	7.35
WATER CONTENT, w, %	**15.81**	**15.1**

RESULTS	
LIQUID LIMIT	**26**
PLASTIC LIMIT	**15**
PLASTICITY INDEX	**11**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

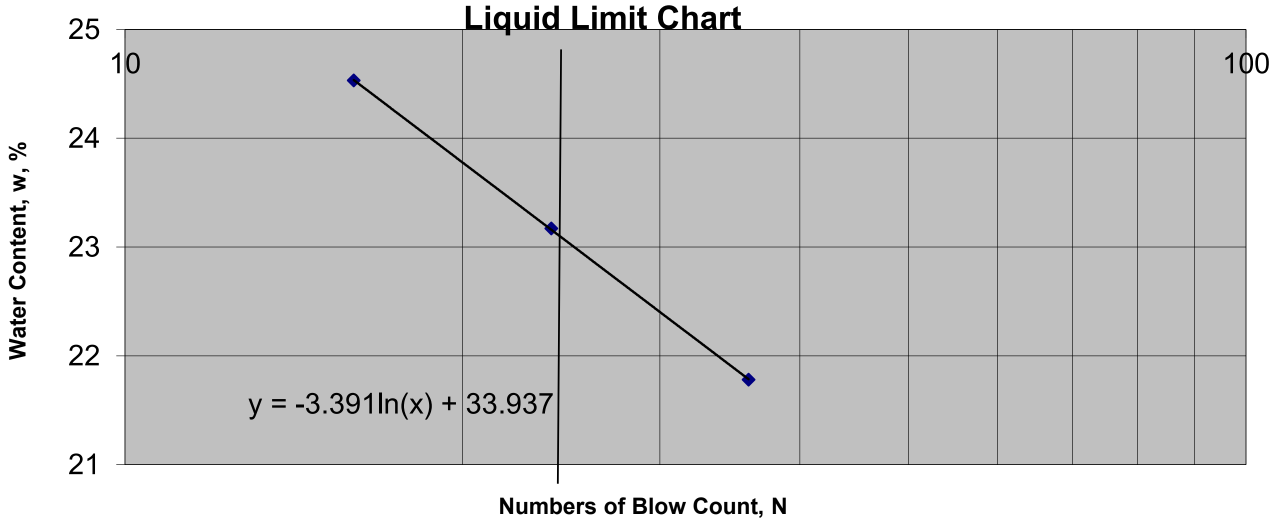
ATTERBERG LIMITS
LS-703/704

CLIENT:	OCH	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	DATE SAMPLED:	06-Jun
LOCATION:	BH6-25 SS6 @ 12'6" - 14'6"	DATE REPORTED:	19-Jun

CAN NO.	**11**	**67**	**87**				
WT. OF CAN	8.65	7.21	7.21				
WT. OF SOIL & CAN	15.86	14.60	13.92				
WT. OF DRY SOIL & CAN	14.44	13.21	12.72				
WT. OF MOISTURE	1.42	1.39	1.20				
WT. OF DRY SOIL & CAN	5.79	6	5.51				
WATER CONTENT, w, %	**24.53**	**23.17**	**21.78**				
NO. OF BLOWS, N	16	24	36				

CAN NO.	**12**	**18**
WT. OF CAN	16.73	20.02
WT. OF SOIL & CAN	25.49	28.99
WT. OF DRY SOIL & CAN	24.34	27.81
WT. OF MOISTURE	1.15	1.18
WT. OF DRY SOIL & CAN	7.61	7.79
WATER CONTENT, w, %	**15.11**	**15.15**

RESULTS	
LIQUID LIMIT	**23**
PLASTIC LIMIT	**15**
PLASTICITY INDEX	**8**



TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

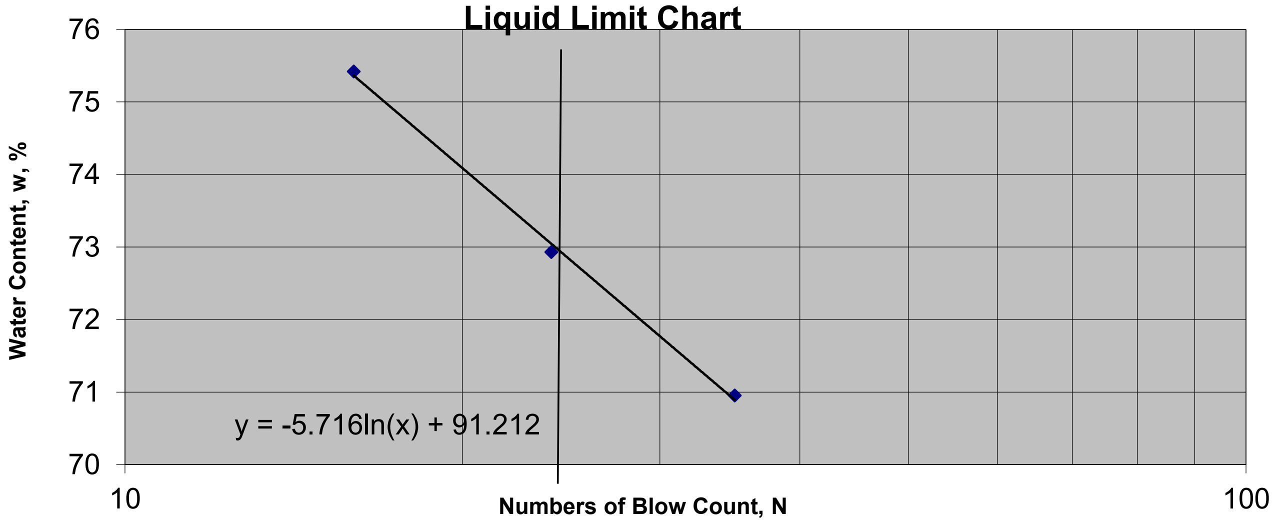
ATTERBERG LIMITS
LS-703/704

CLIENT:	OCH	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	DATE SAMPLED:	06-Jun
LOCATION:	BH7-25 SS4 @ 7'6" - 9'6"	DATE REPORTED:	19-Jun

CAN NO.	**32**	**33**	**35**				
WT. OF CAN	4.36	4.31	4.37				
WT. OF SOIL & CAN	10.64	11.21	9.43				
WT. OF DRY SOIL & CAN	7.94	8.30	7.33				
WT. OF MOISTURE	2.7	2.91	2.10				
WT. OF DRY SOIL & CAN	3.58	3.99	2.96				
WATER CONTENT, w, %	**75.42**	**72.93**	**70.95**				
NO. OF BLOWS, N	16	24	35				

CAN NO.	**1**	**2**
WT. OF CAN	19.85	19.91
WT. OF SOIL & CAN	27.88	27.84
WT. OF DRY SOIL & CAN	25.83	25.84
WT. OF MOISTURE	2.05	2
WT. OF DRY SOIL & CAN	5.98	5.93
WATER CONTENT, w, %	**34.28**	**33.73**

RESULTS	
LIQUID LIMIT	**73**
PLASTIC LIMIT	**34**
PLASTICITY INDEX	**39**

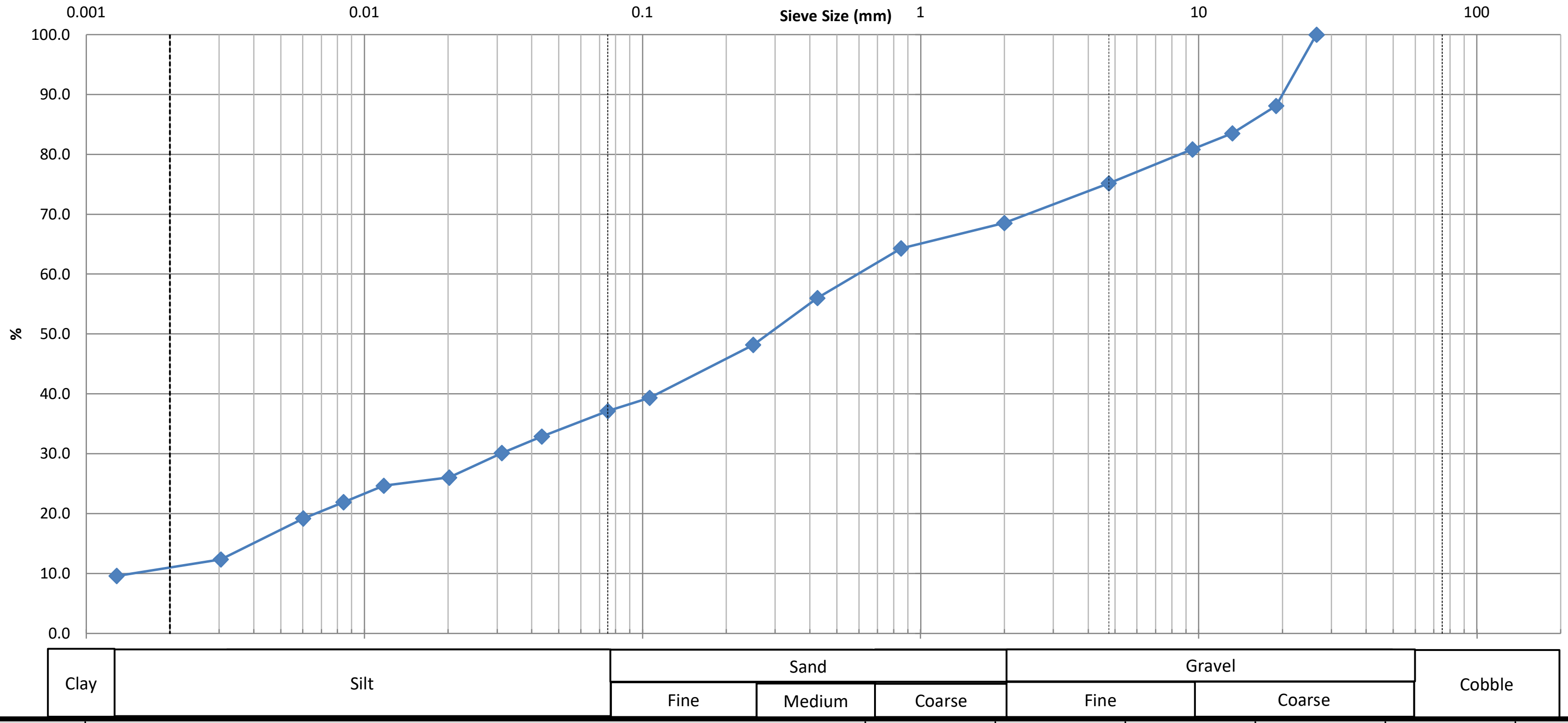


TECHNICIAN: CP	**REVIEWED BY:**	**C. Beadow**	**J. Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	OCH	DEPTH:	10' - 12'	FILE NO:	PG7572
CONTRACT NO.:		BH OR TP No.:	BH3-25 SS5	LAB NO:	60086
PROJECT:	1770 Heatherington Road, Ottawa, Ontario			DATE RECEIVED:	9-Jun-25
				DATE TESTED:	10-Jun-25
DATE SAMPLED:	5-Jun-25			DATE REPORTED:	19-Jun-25
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					11.6%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					24.8	38.0	26.4	10.7		

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	OCH	DEPTH:	10' - 12'	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	BH OR TP No.:	BH3-25 SS5	DATE SAMPLED:	05-Jun-25
LAB No. :	60086	TESTED BY:	D.K	DATE RECEIVED:	09-Jun-25
SAMPLED BY:	-	DATE REPT'D:	19-Jun-25	DATE TESTED:	10-Jun-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
430.9	2.700

INITIAL WEIGHT	50.00	HYGROSCOPIC MOISTURE		
WEIGHT CORRECTED	49.48	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	23.15	AIR DRY	298.40	298.40
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	295.30	295.30
		CORRECTED	0.990	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**51.2**	11.9	**88.1**
13.2	**70.9**	16.5	**83.5**
9.5	**82.5**	19.1	**80.9**
4.75	**107.0**	24.8	**75.2**
2.0	**135.6**	31.5	**68.5**
Pan	**295.3**		
0.850	**3.09**	35.7	**64.3**
0.425	**9.13**	44.0	**56.0**
0.250	**14.86**	51.8	**48.2**
0.106	**21.29**	60.6	**39.4**
0.075	**22.91**	62.9	**37.1**
Pan	**23.15**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	7:16	30.0	6.0	23.0	0.0435	48.0	**32.9**
2	7:17	28.0	6.0	23.0	0.0312	44.0	**30.1**
5	7:20	25.0	6.0	23.0	0.0202	38.0	**26.0**
15	7:30	24.0	6.0	23.0	0.0117	36.0	**24.7**
30	7:45	22.0	6.0	23.0	0.0084	32.0	**21.9**
60	8:15	20.0	6.0	23.0	0.0060	28.0	**19.2**
250	11:25	15.0	6.0	23.0	0.0030	18.0	**12.3**
1440	7:15	13.0	6.0	23.0	0.0013	14.0	**9.6**

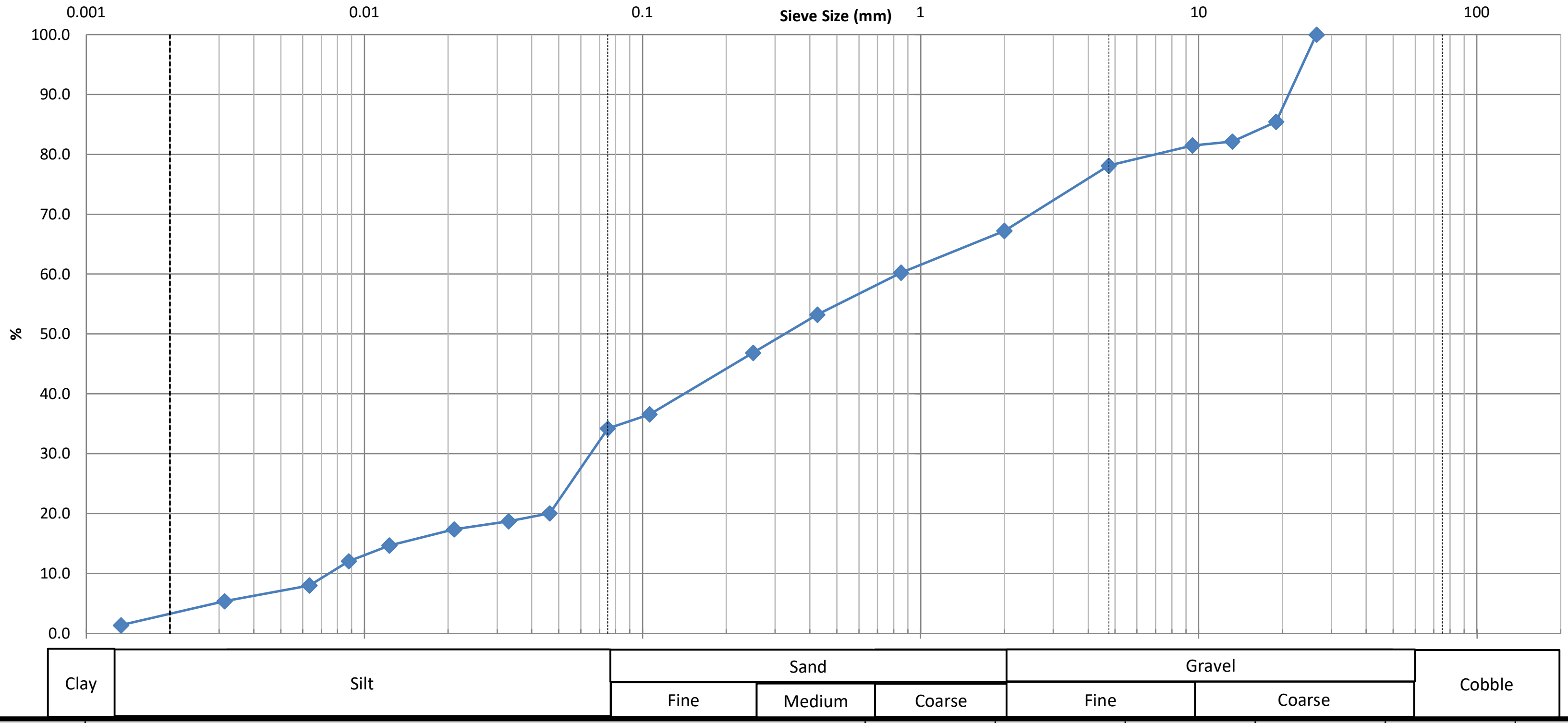
Moisture = 11.6%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	OCH	DEPTH:	12'6" - 14'6"	FILE NO:	PG7572
CONTRACT NO.:		BH OR TP No.:	BH3-25 SS6	LAB NO:	60087
PROJECT:	1770 Heatherington Road, Ottawa, Ontario			DATE RECEIVED:	9-Jun-25
				DATE TESTED:	10-Jun-25
DATE SAMPLED:	5-Jun-25			DATE REPORTED:	19-Jun-25
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					8.7%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					21.9	43.9		31.4	2.8	

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	OCH	DEPTH:	12'6" - 14'6"	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	BH OR TP No.:	BH3-25 SS6	DATE SAMPLED:	05-Jun-25
LAB No. :	60087	TESTED BY:	D.K	DATE RECEIVED:	09-Jun-25
SAMPLED BY:	-	DATE REPT'D:	19-Jun-25	DATE TESTED:	10-Jun-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
487.4	2.700

INITIAL WEIGHT	50.00	**HYGROSCOPIC MOISTURE**		
WEIGHT CORRECTED	49.76	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	24.88	AIR DRY	329.20	329.20
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	327.60	327.60
		CORRECTED	0.995	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**70.8**	14.5	**85.5**
13.2	**87.0**	17.8	**82.2**
9.5	**90.1**	18.5	**81.5**
4.75	**106.6**	21.9	**78.1**
2.0	**159.8**	32.8	**67.2**
Pan	**327.6**		
0.850	**5.19**	39.8	**60.2**
0.425	**10.39**	46.8	**53.2**
0.250	**15.14**	53.1	**46.9**
0.106	**22.78**	63.4	**36.6**
0.075	**24.56**	65.8	**34.2**
Pan	**24.88**		

SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	7:26	21.0	6.0	23.0	0.0464	29.8	**20.0**
2	7:27	20.0	6.0	23.0	0.0330	27.8	**18.7**
5	7:30	19.0	6.0	23.0	0.0210	25.8	**17.4**
15	7:40	17.0	6.0	23.0	0.0123	21.9	**14.7**
30	7:55	15.0	6.0	23.0	0.0088	17.9	**12.0**
60	8:25	12.0	6.0	23.0	0.0063	11.9	**8.0**
250	11:35	10.0	6.0	23.0	0.0031	7.9	**5.3**
1440	7:25	7.0	6.0	23.0	0.0013	2.0	**1.3**

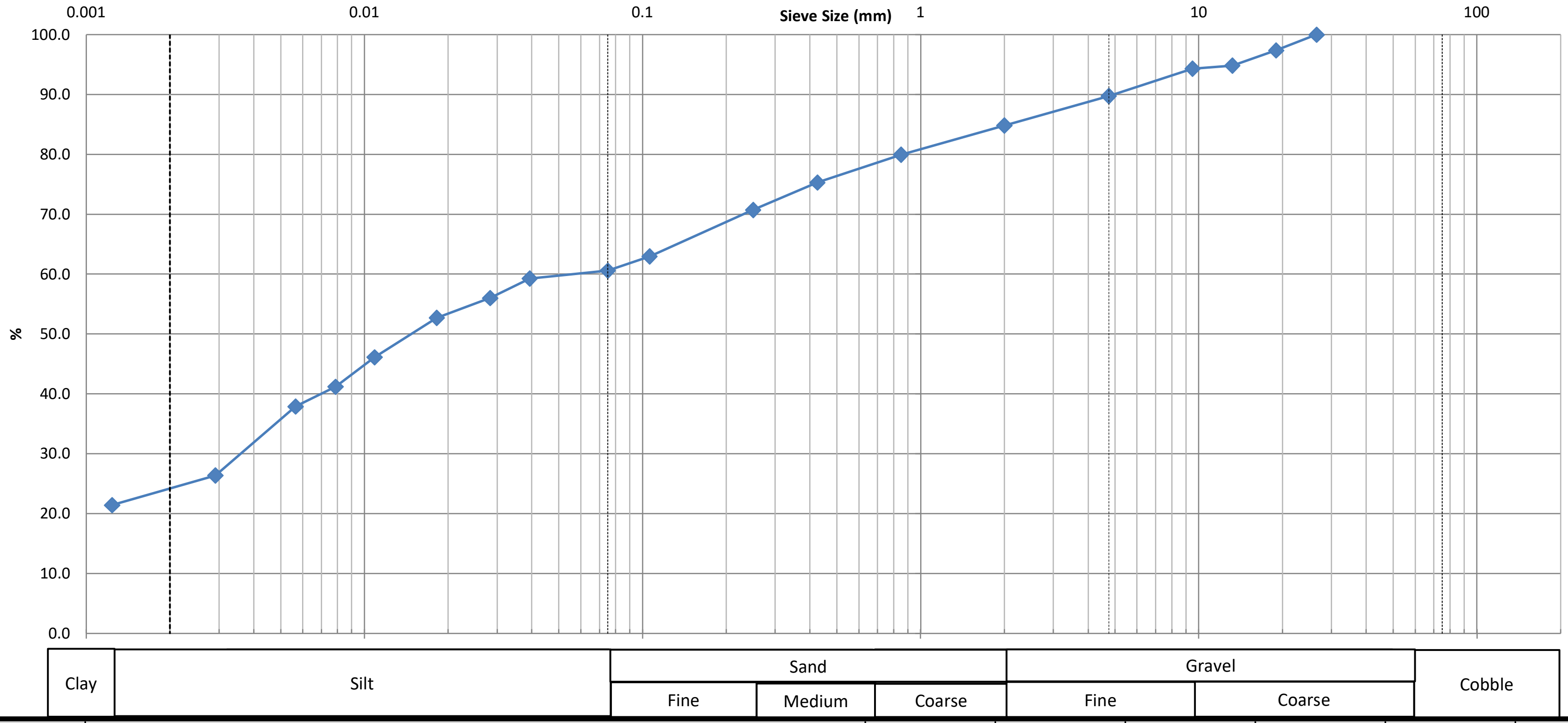
Moisture = 8.7%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	OCH	DEPTH:	12'6" - 14'6"	FILE NO:	PG7572
CONTRACT NO.:		BH OR TP No.:	BH4-25 SS6	LAB NO:	60090
PROJECT:	1770 Heatherington Road, Ottawa, Ontario			DATE RECEIVED:	9-Jun-25
				DATE TESTED:	10-Jun-25
DATE SAMPLED:	5-Jun-25			DATE REPORTED:	19-Jun-25
SAMPLED BY:	-			TESTED BY:	D.K



Identification	**Soil Classification**					**MC(%)**	**LL**	**PL**	**PI**	**Cc**	**Cu**
						21.5%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
					10.2	29.2		36.9		23.7	

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	OCH	DEPTH:	12'6" - 14'6"	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	BH OR TP No.:	BH4-25 SS6	DATE SAMPLED:	05-Jun-25
LAB No. :	60090	TESTED BY:	D.K	DATE RECEIVED:	09-Jun-25
SAMPLED BY:	-	DATE REPT'D:	19-Jun-25	DATE TESTED:	10-Jun-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
429.6	2.700

INITIAL WEIGHT	50.00	HYGROSCOPIC MOISTURE		
WEIGHT CORRECTED	45.84	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	14.42	AIR DRY	397.60	397.60
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	364.50	364.50
		CORRECTED	0.917	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**11.2**	2.6	**97.4**
13.2	**22.2**	5.2	**94.8**
9.5	**24.3**	5.7	**94.3**
4.75	**44.0**	10.2	**89.8**
2.0	**65.1**	15.2	**84.8**
Pan	**364.5**		
0.850	**2.88**	20.0	**80.0**
0.425	**5.62**	24.7	**75.3**
0.250	**8.31**	29.3	**70.7**
0.106	**12.89**	37.0	**63.0**
0.075	**14.29**	39.4	**60.6**
Pan	**14.42**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:38	42.0	6.0	23.0	0.0393	69.9	**59.3**
2	8:39	40.0	6.0	23.0	0.0283	66.0	**56.0**
5	8:42	38.0	6.0	23.0	0.0182	62.1	**52.7**
15	8:52	34.0	6.0	23.0	0.0109	54.4	**46.1**
30	9:07	31.0	6.0	23.0	0.0079	48.5	**41.2**
60	9:37	29.0	6.0	23.0	0.0057	44.7	**37.9**
250	12:47	22.0	6.0	23.0	0.0029	31.1	**26.4**
1440	8:37	19.0	6.0	23.0	0.0012	25.2	**21.4**

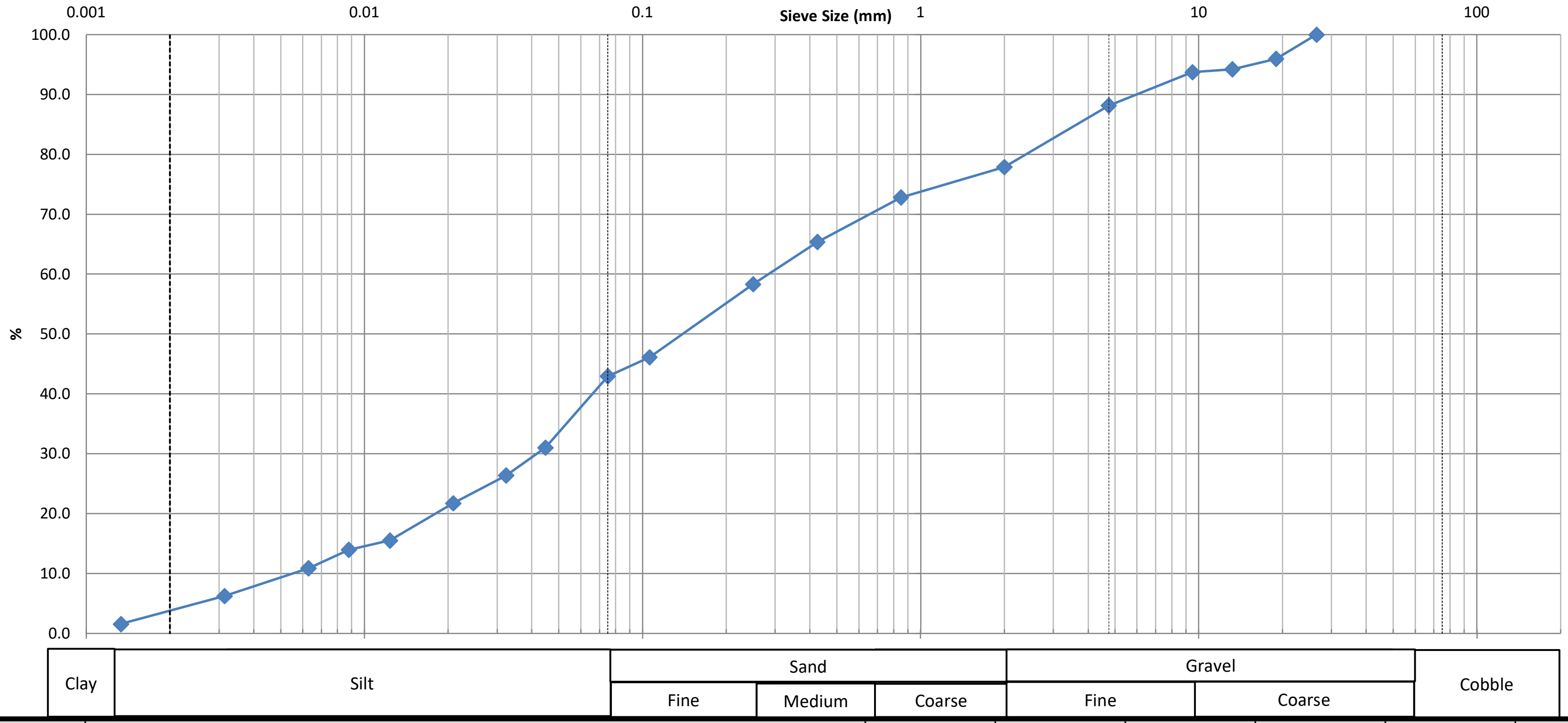
Moisture = 21.5%

REVIEWED BY:	C. Beadow	Joe Forsyth, P. Eng.

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	OCH	DEPTH:	15' - 17'	FILE NO:	PG7572
CONTRACT NO.:		BH OR TP No.:	BH6-25 SS7	LAB NO:	60089
PROJECT:	1770 Heatherington Road, Ottawa, Ontario			DATE RECEIVED:	9-Jun-25
				DATE TESTED:	10-Jun-25
DATE SAMPLED:	5-Jun-25			DATE REPORTED:	19-Jun-25
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					9.4%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)
					11.8	45.2		39.7		3.3

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	OCH	DEPTH:	15' - 17'	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	BH OR TP No.:	BH6-25 SS7	DATE SAMPLED:	05-Jun-25
LAB No. :	60089	TESTED BY:	D.K	DATE RECEIVED:	09-Jun-25
SAMPLED BY:	-	DATE REPT'D:	19-Jun-25	DATE TESTED:	10-Jun-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
409.5	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	49.66	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	22.61	AIR DRY	321.20	321.20
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	319.00	319.00
		CORRECTED	0.993	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**16.4**	4.0	**96.0**
13.2	**23.7**	5.8	**94.2**
9.5	**25.6**	6.3	**93.7**
4.75	**48.4**	11.8	**88.2**
2.0	**90.5**	22.1	**77.9**
Pan	**319.0**		
0.850	**3.25**	27.2	**72.8**
0.425	**8.03**	34.6	**65.4**
0.250	**12.57**	41.7	**58.3**
0.106	**20.41**	53.9	**46.1**
0.075	**22.43**	57.0	**43.0**
Pan	**22.61**		

SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	7:31	26.0	6.0	23.0	0.0448	39.8	**31.0**
2	7:32	23.0	6.0	23.0	0.0323	33.9	**26.4**
5	7:35	20.0	6.0	23.0	0.0209	27.9	**21.7**
15	7:45	16.0	6.0	23.0	0.0124	19.9	**15.5**
30	8:00	15.0	6.0	23.0	0.0088	17.9	**14.0**
60	8:30	13.0	6.0	23.0	0.0063	13.9	**10.9**
250	11:40	10.0	6.0	23.0	0.0031	8.0	**6.2**
1440	7:30	7.0	6.0	23.0	0.0013	2.0	**1.6**

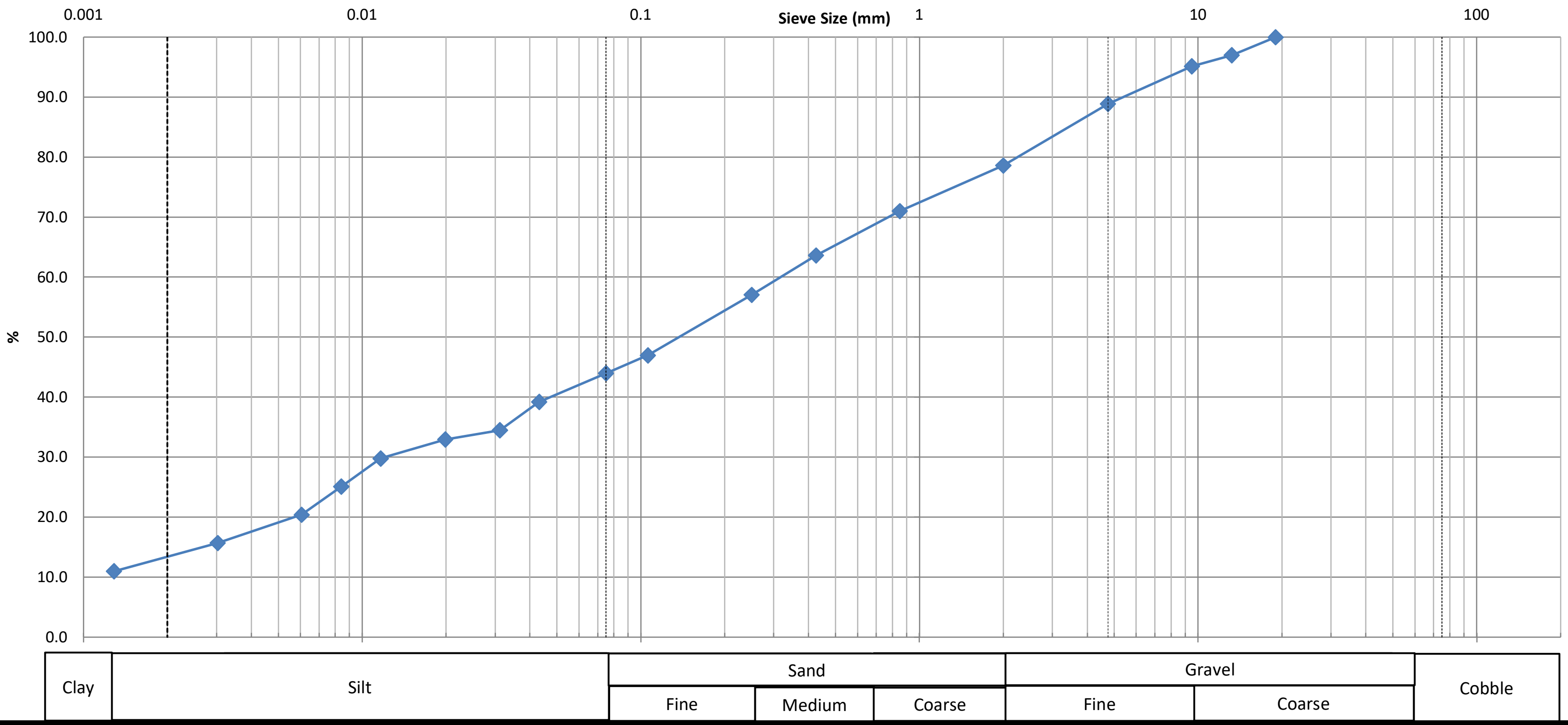
Moisture = 9.4%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	OCH	DEPTH:	15' - 17'	FILE NO:	PG7572
CONTRACT NO.:		BH OR TP No.:	BH7-25 SS7	LAB NO:	60085
PROJECT:	1770 Heatherington Road, Ottawa, Ontario			DATE RECEIVED:	9-Jun-25
				DATE TESTED:	10-Jun-25
DATE SAMPLED:	5-Jun-25			DATE REPORTED:	19-Jun-25
SAMPLED BY:	-			TESTED BY:	D.K



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					10.7%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)	Clay (%)	
					11.1	44.9		31.1	12.9	

Comments:

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

HYDROMETER
LS-702 ASTM-422

CLIENT:	OCH	DEPTH:	15' - 17'	FILE NO.:	PG7572
PROJECT:	1770 Heatherington Road, Ottawa, Ontario	BH OR TP No.:	BH7-25 SS7	DATE SAMPLED:	05-Jun-25
LAB No. :	60085	TESTED BY:	D.K	DATE RECEIVED:	09-Jun-25
SAMPLED BY:	-	DATE REPT'D:	19-Jun-25	DATE TESTED:	10-Jun-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
383.2	2.700

INITIAL WEIGHT	50.00	HYGROSCOPIC MOISTURE		
WEIGHT CORRECTED	49.57	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	22.25	AIR DRY	303.80	303.80
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	301.20	301.20
		CORRECTED	0.991	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19	**0.0**	0.0	**100.0**
13.2	**11.5**	3.0	**97.0**
9.5	**18.6**	4.9	**95.1**
4.75	**42.5**	11.1	**88.9**
2.0	**82.0**	21.4	**78.6**
Pan	**301.2**		
0.850	**4.82**	29.0	**71.0**
0.425	**9.53**	36.4	**63.6**
0.250	**13.71**	43.0	**57.0**
0.106	**20.13**	53.0	**47.0**
0.075	**22.03**	56.0	**44.0**
Pan	**22.25**		

SIEVE CHECK	0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	7:12	31.0	6.0	23.0	0.0431	49.9	**39.2**
2	7:13	28.0	6.0	23.0	0.0312	43.9	**34.5**
5	7:16	27.0	6.0	23.0	0.0199	41.9	**32.9**
15	7:26	25.0	6.0	23.0	0.0116	37.9	**29.8**
30	7:41	22.0	6.0	23.0	0.0084	31.9	**25.1**
60	8:11	19.0	6.0	23.0	0.0061	25.9	**20.4**
250	11:21	16.0	6.0	23.0	0.0030	19.9	**15.7**
1440	7:11	13.0	6.0	23.0	0.0013	14.0	**11.0**

Moisture = 10.7%

REVIEWED BY:	**C. Beadow**	**Joe Forsyth, P. Eng.**

Certificate of Analysis

Report Date: 13-Jun-2025

Client: **Paterson Group Consulting Engineers (Ottawa)**

Order Date: 9-Jun-2025

Client PO: 63285

Project Description: PG7572

	Client ID:	BH3-25 SS5	-	-	-		
	Sample Date:	06-Jun-25 09:00	-	-	-	-	-
	Sample ID:	2524097-01	-	-	-		
	Matrix:	Soil	-	-	-		
	MDL/Units						
Physical Characteristics							
% Solids	0.1 % by Wt.	89.3	-	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.94	-	-	-	-	-
Resistivity	0.1 Ohm.m	15.7	-	-	-	-	-
Anions							
Chloride	10 ug/g	179	-	-	-	-	-
Sulphate	10 ug/g	180	-	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG7572-1 - TEST HOLE LOCATION PLAN

DRAWING PG7572-2 – PERMISSIBLE GRADE RAISE PLAN

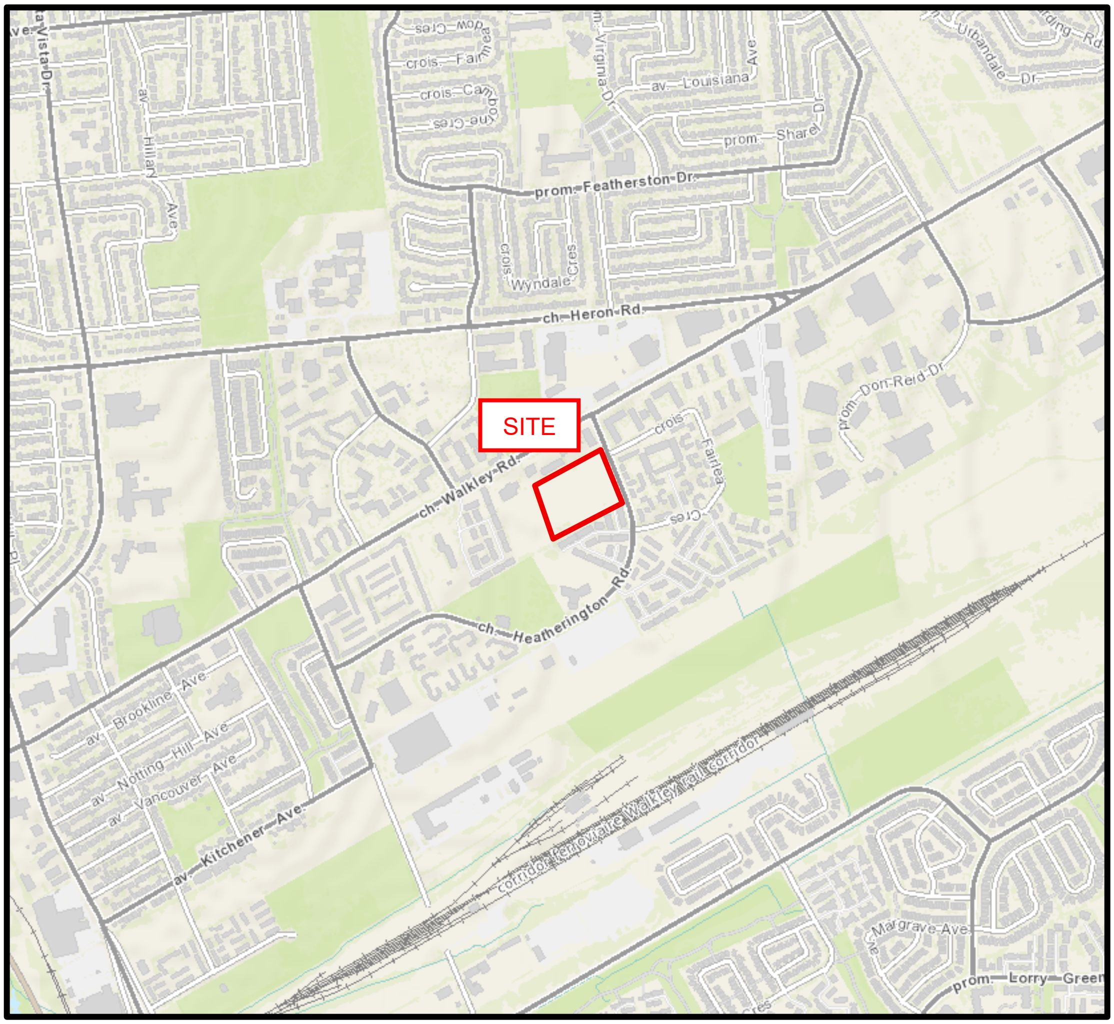
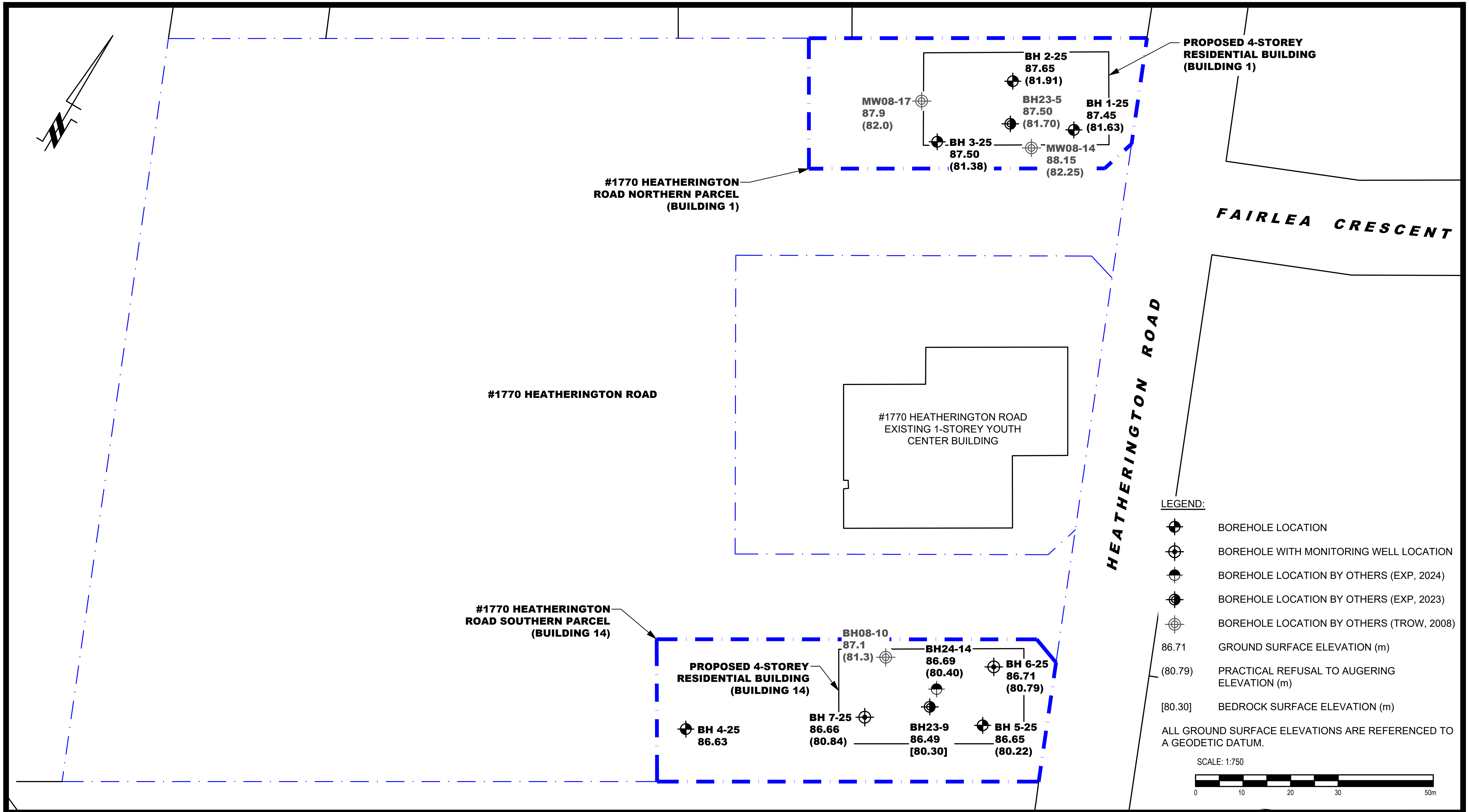


FIGURE 1

KEY PLAN

PATERSON
GROUP



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

NO.	REVISIONS	DATE	INITIAL

OTTAWA COMMUNITY HOUSING CORPORATION

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT - BUILDING 1 & 14
1770 HEATHERINGTON ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:750	Date: 06/2025
Drawn by: NFRV	Report No.: PG7572-1
Checked by: NFRV	Dwg. No.: PG7572-1
Approved by: DP	Revision No.:

11x17

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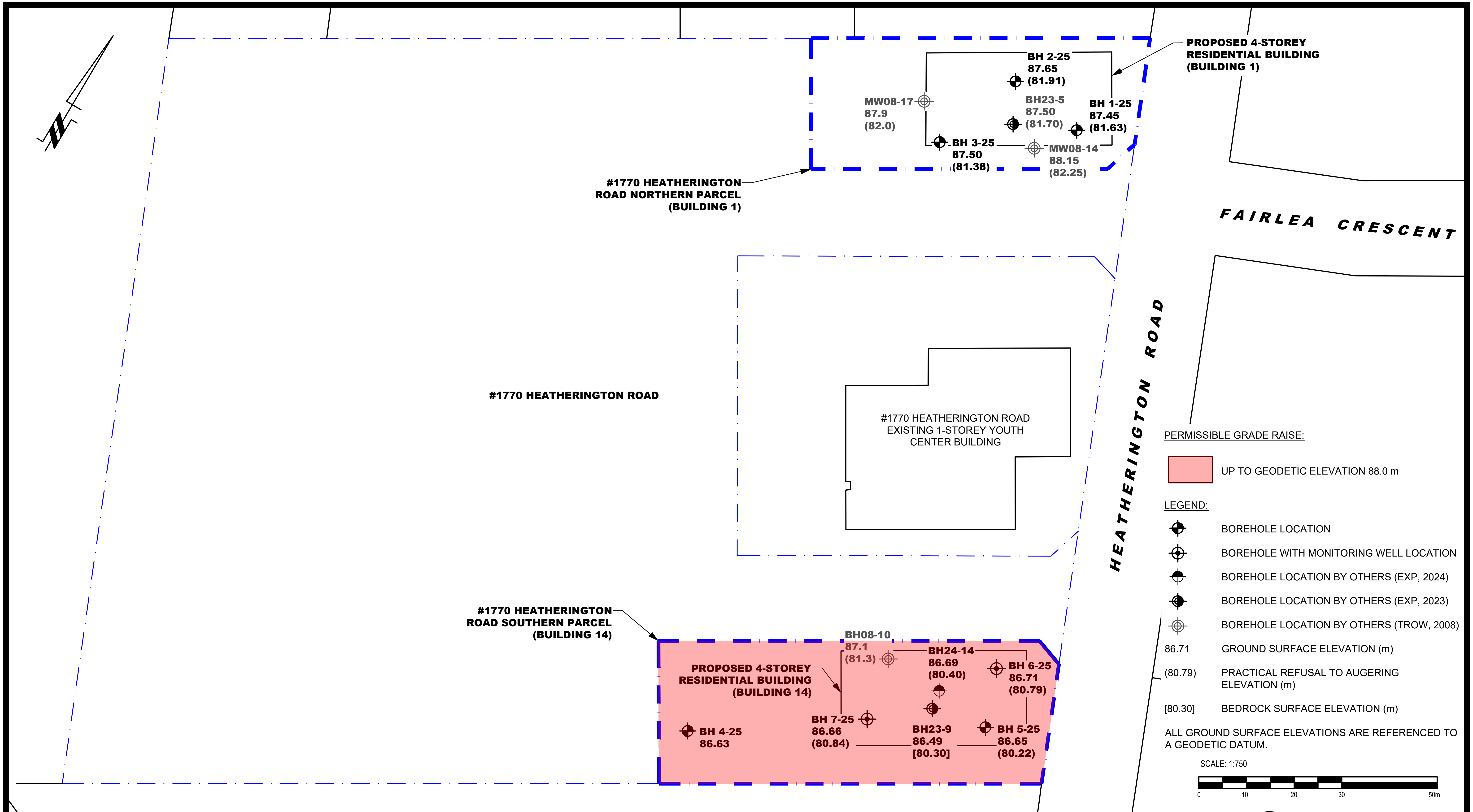

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