PATERSON GROUP

Geotechnical Investigation

Proposed School Development

1010 Somerset Street West
Ottawa, Ontario

Prepared for Conseil des écoles publiques de l'Est de l'Ontario

Report PG7468-1, dated March 21, 2025

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1.0 Introduction

Paterson Group (Paterson) was commissioned by the Conseil des écoles publiques de l'Est de l'Ontario (CEPEO) to conduct a geotechnical investigation for the proposed school development to be located at 1010 Somerset Street West in the City of Ottawa (reference should be made to Figure 1 - Key Plan in Appendix 2 for the general site location).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of test holes.
- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

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

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

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a five-storey school building. One full or partial basement level is anticipated below a portion of the proposed building footprint.

It is further understood that associated landscaped areas, playgrounds and recreational areas, asphalt-paved parking areas and access lanes with landscaped margins are also anticipated throughout the subject site. It is understood the proposed building will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The current geotechnical investigation was conducted on March 12, 2025, and consisted of a total of three (3) boreholes advanced to a maximum depth of 10.0 m below the existing grade.

The test hole locations were distributed in a manner to provide general coverage of the subject site, taking into consideration underground services and available access. The boreholes were drilled using a low-clearance track-mounted auger drilling rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer.

A previous field investigation was undertaken by others within the subject site boundary on September 13 to 15, 2021, and consisted of a total of three (3) boreholes advanced to a maximum depth of 30.5 m below the existing grade. A previous investigation was also undertaken within the subject site boundary by others on October 24 to November 8, 2024, and consisted of advancing four (4) boreholes to a maximum depth of 8.2 m below ground surface.

The locations of the test holes from the current investigation, as well as previous field investigations by others, are depicted on Drawing PG7468-1 - Test Hole Location Plan included in Appendix 2.

Sampling and In Situ Testing

Soil samples were collected from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split spoon (SS) sampler. The bedrock was cored to assess the bedrock type and quality. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores (RC) were placed in cardboard boxes.

All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1. Photographs of the rock core are presented in Appendix 1.The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples.

The SPT results are recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Diamond drilling was completed at boreholes BH 1-25 and BH 2-25 to confirm the bedrock type and quality. A recovery value and a Rock Quality Designation (RQD) value was calculated for each drilled section of bedrock and are presented as RC on the Soil Profile and Test Data sheets in Appendix 1. The recovery value is the ratio of the bedrock sample length recovered over the drilled section length, in percentage.

The RQD value is the total length ratio of intact rock core length more than 100 mm in one drilled section over the length of the drilled section, in percentage. These values are indicative of the quality of the bedrock.

Undrained shear strength testing was carried out at regular depth intervals in cohesive soils, using field vanes. The subsurface conditions observed in the test holes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data Sheets in Appendix 1 of this report. The soil profiles encountered by others are logged on the Borehole Logs by Others included in Appendix 1 of this report.

Groundwater

A flexible polyethylene standpipe was installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program.

Monitoring wells were installed by others as part of the previous investigations undertaken by others. The groundwater observations in the remainder of the boreholes were made in the open boreholes at the time of the previous investigations. The groundwater observations are discussed in Subsection 4.3 of this report and presented on the Soil Profile and Test Data Sheets in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations, and the ground surface elevation at each test hole location, were reported with respect to a geodetic datum. The locations of the test holes, and ground surface elevation at each test hole location, are presented on Drawing PG7468-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Review

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

Paterson understands that soils samples were recovered from the subject site and visually examined to review the results of the field logging by others during the previous investigations. It is understood that all collected soil samples from the 2024 investigation were submitted for moisture content testing. In addition, select samples were submitted for Atterberg Limit tests, Grain-Size testing and consolidation testing. Further, unconfined compressive strength testing was performed on select rock core samples recovered from the previous investigations by others. The results of these tests are described further in Subsection 4.2 of this report and provided in Appendix 1 for further review.

3.4 Analytical Testing

One (1) soil sample from the previous 2024 investigation by others collected from within the subject site boundary 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 Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site consists of an existing two-storey decommissioned municipal building with associated access lanes and parking areas at the northern half of the property. The northeastern section of the subject site consists of grassed land, wile the southeastern section consists of vegetated land. A paved parking area occupies the southern half of the site.

It should be noted that based on the available design drawings, the proposed school building will be located within the southeastern quadrant of the subject site. The area of the proposed building footprint is relatively flat with a gradual increase in grade from west to east of approximately 900 mm.

The site is bordered by Somerset Street West and further by one-storey commercial buildings to the north, a recreational centre (Plant Recreation Centre) to the east, vacant grassed land to the south and by an existing paved parking lot area to the west. Based on Paterson review of historical aerial images, a large two-storey building encompassed the entirety of the southern half of subject site, extending southwards along multiple other property parcels prior to 2015. Arial images taken after 2015 show the southern half of the subject site as it is presently as described above.

4.2 Subsurface Profile

Generally, the subsurface profile at the test hole locations consists of a 0.1 to 0.3 m thick layer of topsoil or a 50 to 100 mm thick layer of asphalt underlain by a layer of fill, further underlain by a deposit of silty clay. A deposit of glacial till was noted below the silty clay layer, which was further underlain by the underlying bedrock formation.

The fill layer was generally observed to consist of silty sand or silty clay with variable amounts of organics, gravel, cobbles, boulders, and construction debris. The fill layer was noted to extend to approximate depths between 1.5 to 2.6 m below the ground surface.

The fill layer was observed to generally be underlain by a deposit of silty clay. The silty clay deposit consists of a hard to very stiff brown silty clay which extended to approximate depths between 3.1 and 5.2 m below the ground surface. The brown silty clay layer was observed to be underlain by a layer of very stiff to stiff grey silty clay which extended to approximate depths between 4.2 and 5.3 m below the ground surface.

The silty clay deposit was observed to be underlain by a deposit of glacial till. The glacial till deposit generally consists of a layer of stiff to firm grey silty clay over a layer of loose to very dense grey silty sand with variable amounts of silt, clay, gravel, cobbles, and boulders. A glacial till layer consisting of compact to dense sandy silt was observed below the clayey and sandy glacial till layers in BH 2-25. The glacial till deposit was observed to extend to approximate depths between 6.9 and 8.6 m below the ground surface.

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

Bedrock

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

Atterberg Limits Testing

Atterberg limits testing was completed on select silty clay 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 1-25	SS6	5.2 – 5.5	18	13	5	22.9	CL-ML
BH 3-25	SS7	5.6 – 6.2	26	17	9	41.6	CL
Atterberg Limits Testing Results Based on the 2024 Investigation by Others							
BH24-5	SS4	2.3 - 2.9	50	23	27	30.7	CH
BH24-6	SS4	3.0 - 3.7	57	23	34	49.4	CH
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; CL-ML: Inorganic Silty Clay							

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 1-25 (SS7)	5.5 – 6.1	15.8	38.4	36.2	9.5
BH 2-25 (SS9)	6.7 – 7.3	10.3	44.1	39.9	5.6
Grain Size Distribution Analysis Based on the 2024 Investigation by Others					
BH24-5 (SS3)	1.5 - 2.1	20.3	39.4	40.3	
BH24-5 (SS4)	2.3 – 2.9	0	10	40	50
BH24-4 (SS7)	6.5	8	33	47	12
BH24-4 (SS9)	7.8	18	57	20	5
BH24-5 (SS8)	7.5	14	49	28	9
BH24-6 (SS5)	4.7	13	24	50	13
BH24-6 (SS8)	6.9 – 7.5	0	95.1	4.9	
BH24-7 (SS7)	6.3	8	54	38	
BH24-7 (SS8)	7.2	18	50	32	

Consolidation Testing

During the previous investigation, consolidation testing was completed on a sample collected within the boundary of the subject site. The results of the consolidation test from the previous investigation are presented in Table 3 and presented on the Consolidation Testing Results sheets in Appendix 1. The value for p'c is the preconsolidation pressure. The value of p'c, is determined using standard engineering testing procedures and are estimates only given the natural variations of the in-situ soils and limited sample size.

Table 3 – Summary of Consolidation Test Results							
Borehole	**Depth (m)**	**Bulk Unit Weight (kN/m³)**	**Initial Void Ratio e_o**	**Cc**	**Cr**	**P'_o (kPa)**	**p'_c (kPa)**
BH24-7 (ST-4)	3.8 – 4.4	17.1	1.33	1.15	0.02	70	400

4.3 Groundwater

Groundwater levels were measured in the standpipe piezometers on March 20, 2025, for the current investigation. Groundwater levels measured by others as part of previous investigations have been provided in Table 4. The measured groundwater levels are presented on the Soil Profile and Test Data sheets in Appendix 1, and in Table 4 below.

Table 4 – 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	59.63	5.79	53.84	March 20, 2025
BH 2-25	Piezometer	59.96	6.24	53.72	March 20, 2025
BH 3-25	Piezometer	59.57	4.68	54.89	March 20, 2025
Groundwater Levels Based on the 2024 Investigation by Others					
BH24-5	Monitoring Well	59.69	4.20	55.49	October 28, 2024
MW21-20S	Monitoring Well	60.09	5.41	54.68	October 28, 2024
MW21-13	Monitoring Well	59.18	4.60	54.58	October 28, 2024
MW21-15	Monitoring Well	59.80	6.6	53.20	October 28, 2024

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.

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

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 school building may be founded on conventional spread footings placed on the in-situ, undisturbed, very stiff silty clay or an approved fill 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 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 fill is encountered below footings for the proposed building, it is recommended to be sub-excavated a minimum depth of 450 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 450 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.

It is recommended that where the fill is approved to remain in place below the proposed building's footings that the approved surface be raised to the design founding elevation for the overlying footing with OPSS Granular A placed in maximum 300 mm thick loose lifts and compacted to a minimum of 98% of the materials standard Proctor maximum dry density (SPMDD) (i.e., provide a minimum 500 mm thick layer of OPSS Granular A below all footings underlain by a Paterson-approved fill bearing surface).

If the native overburden is encountered within the sub-excavation, the sub-excavation may terminate upon suitably native, undisturbed, in-situ soils described in Section 5.3 of this report and as reviewed and approved by Paterson field personnel (i.e., sub-excavations less than 450 mm below USF may be terminated before 450 mm if suitable native subsoils are encountered within the recommended sub-excavation depth).

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 and who are experienced in reviewing the placement of soil fill in this manner. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as CCW MiraDRAIN 2000 or Delta-Teraxx.

Protection of Subgrade (Conventional Spread Footings)

It is anticipated that the clay subgrade soils will become readily disturbed by construction traffic. Therefore, it is recommended that a minimum 50 mm thick mud slab layer be placed over the prepared bearing medium for all footings once the bearing surface has been reviewed and approved by Paterson personnel.

The mud slab is recommended to consist of a minimum 15 MPa (28-day compressive strength) concrete, extend a minimum of 150 mm beyond all faces of the overlying footing and should not be placed until the bearing medium has been reviewed and approved at the time of construction by Paterson field personnel.

Protection of Subgrade (Raft Foundation)

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 buildings be supported on a deep foundation, the use of heavy equipment would be required to install the piles and/or 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 basement slab.

5.3 Foundation Design

Bearing Resistance Values – Conventional Spread Footings

Footings placed on an undisturbed, soil bearing surface or surface sounded bedrock can be designed using the following bearing resistance values provided in Table 5 below. 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.

Table 5 - Bearing Resistance Values		
Bearing Surface	**Bearing Resistance Values (kPa)**	
	SLS	**ULS**
Very Stiff to Stiff Brown Silty Clay	150	225
Stiff to Firm Grey Silty Clay	75	110
Compact to Dense Glacial Till	200	300
Approved Proof-Rolled Fill	100	150
Note: Strip and pad footings, up to 3 and 5 m wide, respectively, can be designed using the bearing resistance values provided for an undisturbed, brown silty clay or glacial till bearing surface. Strip and pad footings, up to 3 m wide can be designed using the bearing resistance values provided for an undisturbed, grey silty clay or approved proof-rolled fill bearing surface.		

Raft Foundation

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 school building will include a partial basement level. As such, the base of the raft foundation would be placed at different elevations based on the presence or lack thereof of a basement level.

For design purposes, it was assumed that the base of the raft foundation of the partial basement level would be located at an approximate depth of 3 to 4 m below the existing ground surface. The base of the raft foundation where no basement level would be present was assumed to be located at an approximate depth of 1 m below the existing ground surface.

However, 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.

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 of the partial basement level, a bearing resistance value at SLS (contact pressure) of **200 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff brown or grey silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on one partial basement level, it is expected that the raft foundation will be installed on the brown or grey silty clay deposit. The modulus of subgrade reaction was calculated to be **8.0 MPa/m** for a contact pressure of **200 kPa**. The raft foundation design is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

For the raft slab foundation without a basement level, a bearing resistance value at SLS (contact pressure) of **175 kPa** will be considered acceptable for a raft supported on the undisturbed, stiff brown or grey silty clay. The factored bearing resistance (contact pressure) at ULS can be taken as **225 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance values at ULS.

Based on one partial basement level, it is expected that the raft foundation will be installed on the brown or grey silty clay deposit. The modulus of subgrade reaction was calculated to be **7.0 MPa/m** for a contact pressure of **175 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

A deep foundation method, such as end bearing piles, may 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 6. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values.

Re-striking of all piles, at least once, will also be required after at least 48 hours have elapsed since initial driving. A full-time field review program should be conducted by Paterson field personnel during the pile driving operations to record the pile lengths, ensure that the refusal criteria is met and that piles are driven within the location tolerances (within 75 mm of proper location and within 2% of vertical). Paterson may undertake dynamic pile testing at the time of pile driving and planning.

Table 6 - 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 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, where utilized. Unit weights of materials are provided in Table 7.

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

As noted 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.

Table 7 - 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 under footing-supported structures 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 and our experience with the local silty clay deposit, a permissible grade raise restriction of **2 m** is recommended in the immediate area of settlement sensitive structures.

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

5.4 Design for Earthquakes

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

Field Program

The seismic array was located as presented in Drawing PG7418-1 - Test Hole Location Plan attached to the present report. Paterson field personnel placed 24 horizontal 4.5 Hz geophones mounted to the surface by means of two 75 mm ground spike attached to the geophone land case. The geophones were spaced at 2 m intervals and were connected by a geophone spread cable to a Geode 24 Channel seismograph.

The seismograph was also connected to a laptop computer and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) to eight (8) times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 14.0, 3.0 and 2.0 m away from the first and last geophone, and at the centre of the geophone array.

Data Processing and Interpretation

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

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

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

Based on our testing results, the average overburden shear wave velocity is **322 m/s**, while the bedrock shear wave velocity is **2,201 m/s**. Assuming the proposed structure will be founded at approximate elevation 57.5 m, approximately 6 m of overburden will be present below the foundation.

Based on the above considerations, the V_{s30} was calculated using the standard equation for average shear wave velocity and as presented below:

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

$$V_{s30=} \frac{30\ m}{\left(\frac{6\ m}{322\ m/s} + \frac{24\ m}{2{,}201\ m/s}\right)}$$

$$V_{s30=}\ 1{,}016\ m/s$$

Based on the results of the shear wave velocity testing, the average shear wave velocity, V_{s30}, for the proposed building is **1,016 m/s**. Therefore, as per OBC 2024 a **Site Designation X_{760}** is applicable for the proposed for the design of the proposed structure founded at an approximate elevation of 57.5 m within the subject site.

Based on Paterson's review of the in-situ soils compactness and stiffness for non-cohesive and cohesive soils, respectively, the soils underlying the subject site are not considered susceptible to liquefaction.

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. 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 Basement Slab

Where a raft slab is utilized, a granular layer of OPSS Granular A will be required to allow for the installation of sub-floor services above the raft slab foundation. The thickness of the OPSS Granular A crushed stone will be dependent on the piping requirements.

For buildings founded on footings or piles, it is recommended that the upper 200 mm of subfloor fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of the construction, a sub-floor drainage system, consisting of lines of perforated drainage pipe sub-drains connected to a positive outlet, should be provided in the clear stone under the lower basement floor.

5.7 Basement Wall

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

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

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

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

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

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

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to the latest revision of the Ontario Building Code. Note that the vertical seismic coefficient is assumed to be zero. The earth force component (P_o) under seismic conditions can be calculated using:

$P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

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

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

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per the latest revision of the Ontario Building Code.

5.8 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 Table 8 and Table 9 below.

Table 8 – Recommended Pavement Structure – Light Vehicle Parking and Playground 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 9 – Recommended Pavement Structure – Local Roadways and Bus Lanes, Access Lanes and Heavy Vehicle Parking	
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.

Temporary Access Roads and Construction Traffic

Paterson anticipates that the earthworks contractor will require several haul roads, staging areas and other temporary access lanes to facilitate construction traffic. Paterson also anticipates construction traffic will be directed over unpaved access paths constructed using the base and subbase layers identified in the above-noted tables and will be used throughout the duration of the construction phase.

Omitting the asphalt layer, the above-noted pavement designs are not considered suitable to support temporary construction traffic without requiring additional measures to remediate the proposed base and subbase layers to accommodate the placement of asphalt to complete the pavement design.

Therefore, provisions should be carried to either reinstate temporary construction access and haul roads prior to placing asphalt or improve the durability of the temporary unpaved construction access and haul roads to minimize additional efforts for preparing the base course for the placement of asphalt once construction traffic would no longer be required.

Examples of scenarios that would require these provisions would consist of areas which construction traffic results in rutting and compromising subgrade soils, placement of subbase layers directly over subgrade shortly following periods of spring thaw, snowmelt and rainfall events or over service trenches that may consist of poorly compacted backfill.

For planning purposes, temporary construction haul roads and working pads should be planned to be 600 mm of crushed stone consisting of a 500 mm of a combination of OPSS Granular B Type I or Type II crushed stone and/or blast-rock covered with a minimum 50 to 100 mm thick layer of OPSS Granular B Type II or OPSS Granular A crushed stone (to provide suitable surface for vehicle tires) over a Paterson-reviewed and -approved subgrade.

These types of roads should also be underlain by a non-woven geotextile layer, such as Terraifx 200R, where they would be integrated into the final pavement structure and accommodate the placement of asphalt to minimize pumping of fines into the subbase layer. Cow-pathing site-generated soil may also be considered to provide suitable haul and access roads.

Temporary access roads that will not support heavy truck traffic (i.e., conventional light-duty vehicles only) may be prepared using a minimum of 150 mm of OPSS Granular A and 400 mm of OPSS Granular B Type II crushed stone. However, provisions should be carried to provide a non-woven geotextile separation layer, such as Terrafix 200R, over the subgrade soils to lessen the amount of fines that migrate into the subbase layers in response to a combination of construction traffic and seasonal fluctuations in the subgrades performance. Provisions should also be carried to scarify and replace the upper 100 to 150 mm of these areas with clean OPSS Granular A crushed stone prior to placing asphalt.

Provisions should also be carried by the earthworks contractor to suitably compact trench backfill placed over services when reinstating servicing trenches below areas proposed to support paved areas.

Since it is anticipated this material would consist of workable brown silty clay or silty sand fill (and not wet, non-workable grey silty clay) it would be recommended to place this material in maximum 400 mm thick loose lifts compacted using a suitably sized vibratory sheepsfoot roller making several passes under the supervision of Paterson field personnel. The subgrade surface is also recommended to be provided with a layer of bi-axial geogrid, such as Terrafix TBX2500, to improve the stiffness of the reinstated trench backfill subgrade for supporting the final pavement structures.

These efforts would be reviewed, approved and advised upon by Paterson field staff during the construction program. Further, Paterson should review design, tender and construction documents associated with temporary and permanent pavement design throughout those phases of the project.

Pavement Joint Tie-in

Where the proposed pavement structure meets an existing pavement structure, such as the 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 hardscaping is anticipated around the perimeter of the proposed structure, it is recommended to implement a perimeter foundation drainage system around the entire building perimeter. In areas where hard-scaping or pavement structures will abut the building footprint, 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 pipe should have a positive outlet, such as a gravity connection to the storm sewer.

The perimeter drainage pipe should be placed against the structure and with the invert of the pipe placed a minimum of 600 mm below proposed finished grade (i.e., within the subgrade layer and below the crushed stone fill for the hardscaping) and upon either in-situ soils and/or Paterson-reviewed and-approved 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 hardscaping, 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.

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 they are known and able to be reviewed in further detail from a geotechnical perspective.

6.2 Protection Against Frost Action

Foundation Structures

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 eastern side of the proposed structure.

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 (i.e. along the eastern section of the proposed structure and the eastern property line). 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 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 earth pressures acting on the shoring system may be calculated using the parameters provided in Table 10.

Table 10 - 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 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 loose silty clay 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.

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

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 springling 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 direct water away from all subgrades, regardless of the source, to prevent disturbance to the founding medium.

Groundwater Control for Building Construction

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR).

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 Considerations

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 by Paterson and by others for the recovered silty clay samples at selected locations throughout the subject site. The testing results 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 that a tree to foundation setback equal to the full mature height of the tree can be provided.
- ❑ 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 conditions noted below are met.
- ❑ A small tree must be provided with a minimum of 25 m^3 of available soils volume while a medium tree must be provided with a minimum of 30 m^3 of available soil volume, as determined by the Landscape Architect. The developer is to ensure that the soil is generally un-compacted when backfilling in street tree planting locations.
- ❑ The tree species must be small (mature tree height up to 7.5 m) to medium size (mature tree height 7.5 m to 14 m) as confirmed by the Landscape Architect.
- ❑ 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 Grading Plan.

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

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 Conseil des écoles publiques de l'Est de l'Ontario, 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.

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

Drew Petahtegoose, P.Eng.

LICENSED PROFESSIONAL ENGINEER
March 21, 2025
D. J. PETAHTEGOOSE
100568013
PROVINCE OF ONTARIO

Report Distribution:

- ❑ Conseil des écoles publiques de l'Est de l'Ontario (Email Copy)
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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

BOREHOLE LOGS BY OTHERS

ROCK CORE PHOTOGRAPHS

ATTERBERGS TESTING RESULTS

ATTERBERGS TESTING RESULTS BY OTHERS

GRAIN-SIZE TESTING RESULTS

GRAIN-SIZE TESTING RESULTS BY OTHERS

CONSOLIDATION TESTING RESULTS BY OTHERS

ANALYTICAL TESTING RESULTS BY OTHERS

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1010 Somerset Street West, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 366205.15 **NORTHING:** 5029970.50 **ELEVATION:** 59.63

PROJECT: Proposed School Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 12, 2025

FILE NO. : **PG7468**

HOLE NO. : **BH 1-25**

SAMPLE DESCRIPTION

GROUND SURFACE

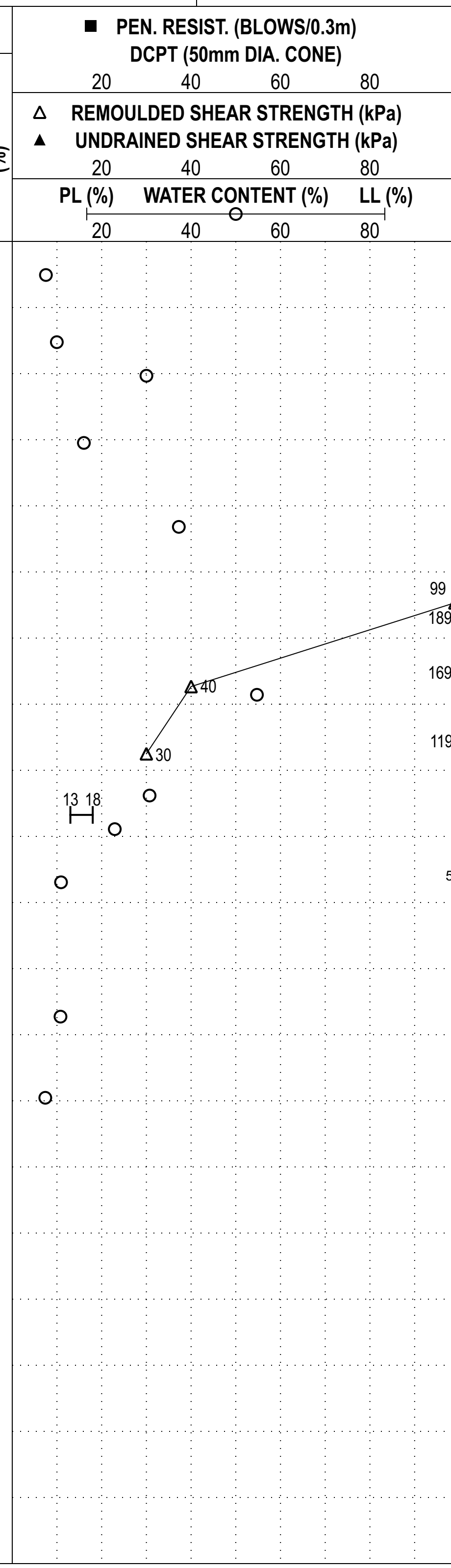
ASPHALT	0.05m [59.58m]
FILL: Brown silty sand, with gravel and crushed stone	1.07m [58.56m]
FILL: Brown silty clay, trace sand and gravel	2.21m [57.42m]
Very stiff, brown **SILTY CLAY** - Grey by 3.8 m depth	5.03m [54.60m]
GLACIAL TILL: Firm, grey silty clay, trace gravel	5.51m [54.12m]
GLACIAL TILL: Loose to dense grey silty sand, trace clay, with gravel, cobbles and boulders	7.47m [52.16m]
GLACIAL TILL: Dense to very dense, grey silty sand, with gravel, cobbles and boulders	8.59m [51.04m]
BEDROCK: Good to excellent quality limestone, with interbedded black shale	10.03m [49.60m]

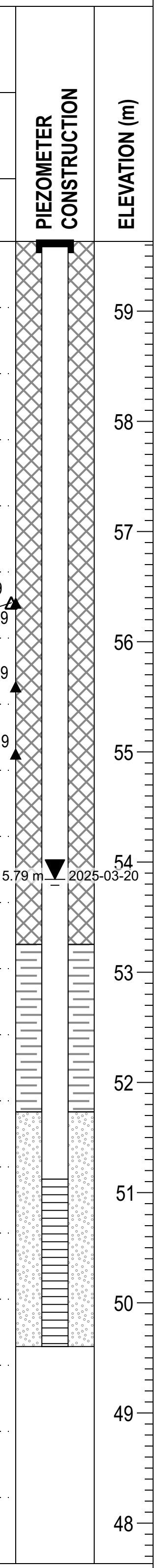
End of Borehole

(GWL at 5.79 m depth - March 20, 2025)

SAMPLE

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			8
SS 2	58	23-24-7-5 31	10 30
SS 3	50	3-5-6-7 11	16
SS 4	75	2-3-3-4 6	37
SS 5		P	55
SS 6	100	1-1-2-1 3	31 23
SS 7	100	2-2-4-8 6	11
SS 8	17	3-4-5-6 9	11
SS 9	33	4-16-17-14 33	7
SS 10	14	14-25-50-/ 75/0.2	
RC 1	100	RQD 80	
RC 2	100	RQD 100	





DEPTH (m): 1 2 3 4 5 6 7 8 9 10 11 12

STRATA PLOT

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1010 Somerset Street West, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 366242.53 **NORTHING:** 5029950.03 **ELEVATION:** 59.96

PROJECT: Proposed School Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 12, 2025

FILE NO. : PG7468

HOLE NO. : BH 2-25

[redacted] GROUND SURFACE

SAMPLE DESCRIPTION	DEPTH (m)	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
0.10m [59.86m]		AU 1			27
FILL: Brown silty clay, trace sand and gravel	1	SS 2	58	2-2-2-3 4	22
1.45m [58.51m]					
FILL: Brown silty sand, with gravel, trace clay		SS 3	58	2-4-10-3 14	9 / 30
1.98m [57.98m]	2				
Hard to very stiff, brown **SILTY CLAY**		SS 4	100	6-6-6-6 12	37
	3	SS 5	100	P	40
	4				
4.22m [55.74m]					
GLACIAL TILL: Stiff to firm, grey silty clay, with gravel, trace sand		SS 6	12	8-5-4-3 9	25
4.93m [55.03m]	5				
GLACIAL TILL: Loose to dense grey silty sand, trace clay, with gravel, cobbles and boulders		SS 7	58	5-2-4-8 6	10
	6	SS 8	42	11-19-18-11 37	17
	7	SS 9	83	14-7-6-7 13	9
7.47m [52.49m]					
GLACIAL TILL: Compact to dense grey sandy silt, with gravel, cobbles and boulders		SS 10	58	7-9-25-45 34	8
8.26m [51.70m]	8				
BEDROCK: Good quality limestone, with interbedded black shale		RC 1	100	RQD 77	
	9	RC 2	100	RQD 97	
10.01m [49.95m]	10				
End of Borehole					
(GWL at 6.24 m depth - March 20, 2025)	11				
	12				

STRATA PLOT

■ PEN. RESIST. (BLOWS/0.3m)
DCPT (50mm DIA. CONE)
20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa)
20 40 60 80

PL (%) WATER CONTENT (%) LL (%)
20 40 60 80

249
199
△60

PIEZOMETER CONSTRUCTION

6.24 m 2025-03-20

ELEVATION (m): 59, 58, 57, 56, 55, 54, 53, 52, 51, 50, 49, 48

DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

1010 Somerset Street West, Ottawa, Ontario

PATERSON GROUP

COORD. SYS.: MTM ZONE 9 **EASTING:** 366232.06 **NORTHING:** 5029978.83 **ELEVATION:** 59.57

PROJECT: Proposed School Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: March 12, 2025

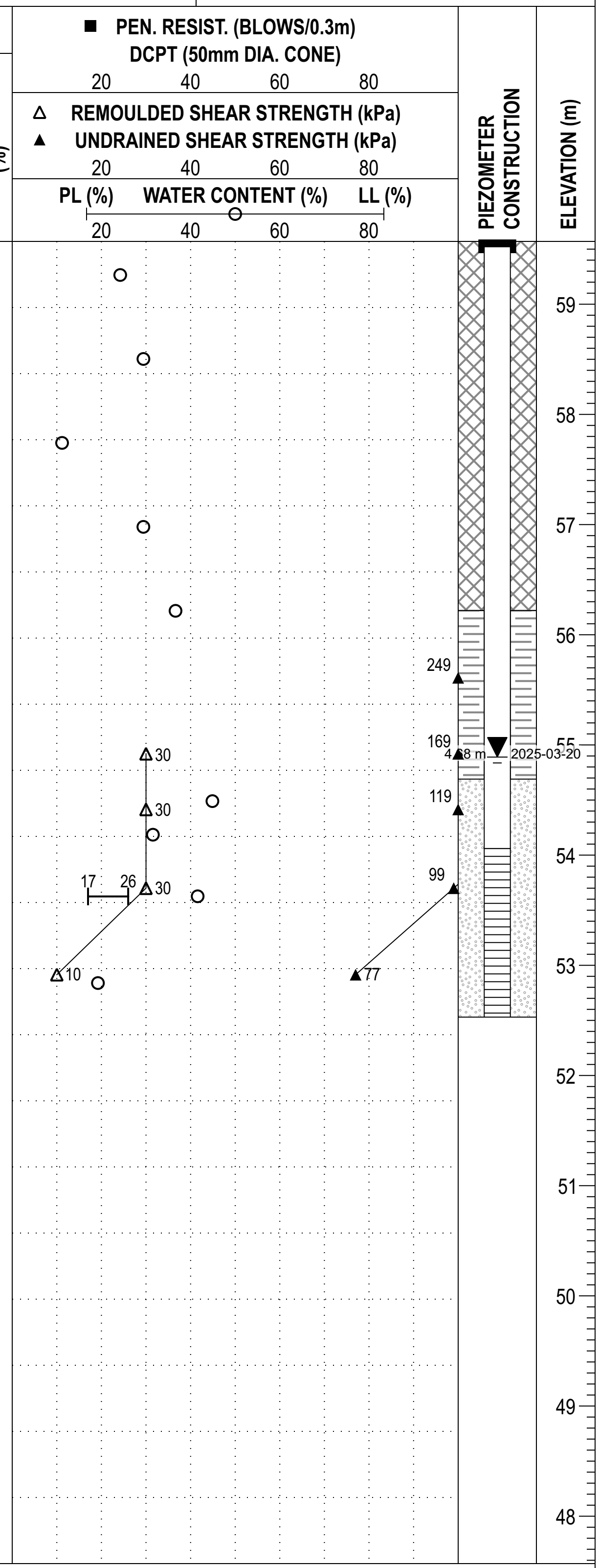
FILE NO. : PG7468

HOLE NO. : BH 3-25

SAMPLE DESCRIPTION (GROUND SURFACE)	DEPTH (m)	SAMPLE TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
TOPSOIL — 0.10m [59.47m]		AU 1			24
FILL: Brown silty clay, with gravel, trace sand, occasional boulders	1	SS 2	58	1-2-4-4 6	29
	2	SS 3	33	10-28-9-5 37	11
2.21m [57.36m] — Hard to very stiff, brown **SILTY CLAY**		SS 4	100	5-6-8-8 14	29
	3	SS 5	100	4-5-5-5 10	37
	4				
5.23m [54.34m] — **GLACIAL TILL:** Stiff, grey silty clay, trace gravel	5	SS 6	100	P	45 32
- Some sand by 5.6 m depth	6	SS 7	100	P	42
- Some cobbles and boulders by 6.4 m depth		SS 8	17	P	19
7.04m [52.53m] — End of Borehole	7				

End of Borehole

(GWL at 4.68 m depth - March 20, 2025)



DISCLAIMER: THE DATA PRESENTED IN THIS SHEET IS THE PROPERTY OF PATERSON GROUP AND THE CLIENT FOR WHOM IT WAS PRODUCED. THIS SHEET SHOULD BE READ IN CONJUNCTION WITH ITS CORRESPONDING REPORT. PATERSON GROUP IS NOT RESPONSIBLE FOR THE UNAUTHORIZED USE OF THIS DATA.

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

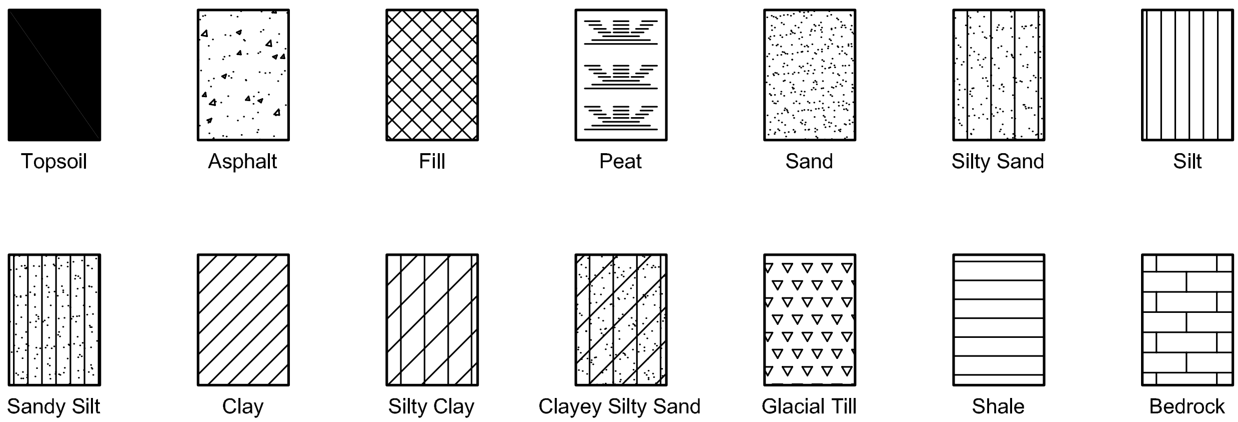
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

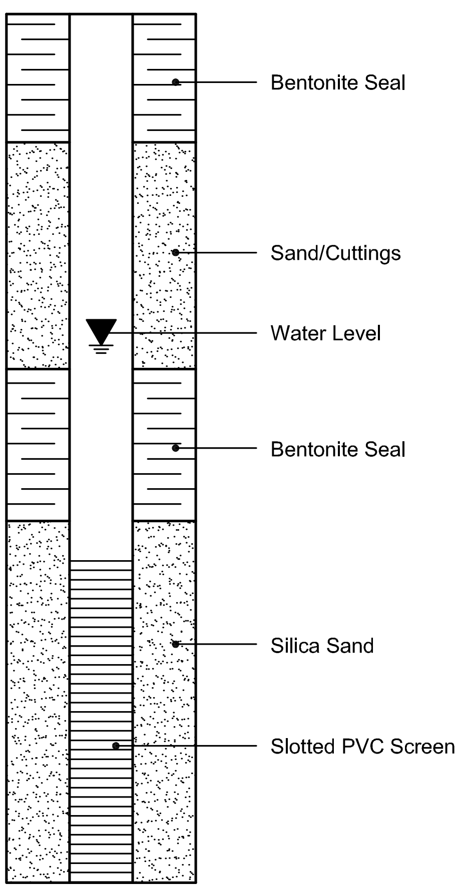
SYMBOLS AND TERMS (continued)

STRATA PLOT

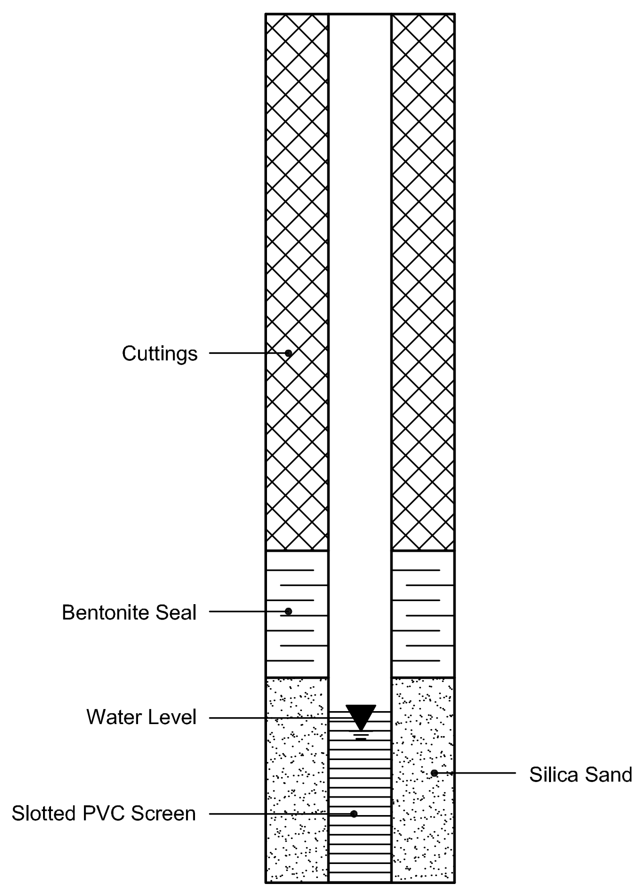


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Stantec

BOREHOLE RECORD

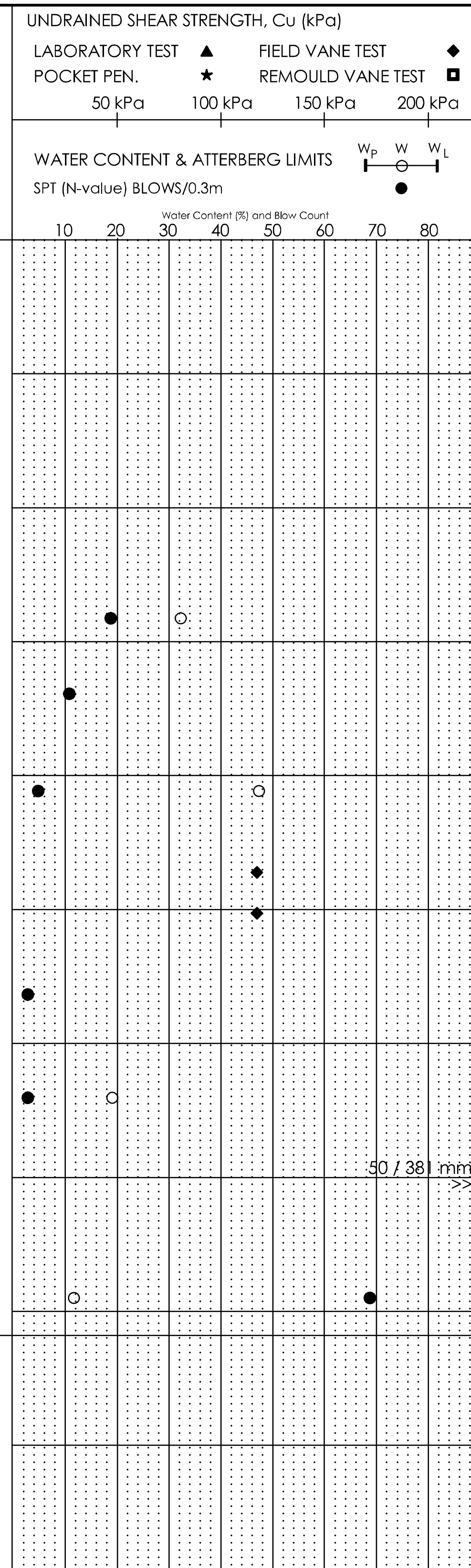
BH24-4

CLIENT: **City of Ottawa**
PROJECT: **1010 Somerset**
LOCATION: **1010 Somerset, Ottawa, ON**
DATE BORED: **11/08/2024**

BH COORDINATES: 5028443.7N 444015.4E
WATER LEVEL: **N/A**

PROJECT NO.: **160402067**
BH ELEVATION: **59.41m**
DATUM: **Geodetic**

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	SAMPLE TYPE	SAMPLE NUMBER	RECOVERY (mm) or TCR %	N-VALUE or RQD %	OTHER TESTS / REMARKS
0	59.4	TOPSOIL - silty sand containing rootlets and organics	GS	1	-	-	
	59.3	FILL: grey sandy clay with gravel containing cobbles and boulders - moist	GS	2	-	-	
2	57.4	Very stiff brown lean CLAY (CL) - weathered crust - moist	SS	3	559	19	
	56.3	Firm to very stiff grey lean CLAY to CLAY (CL-CH) - moist	SS	4	508	11	
			SS	5	610	5	
			SV	-	-	-	
			SV	-	-	-	
	54.0	Very loose to dense grey sandy silt (ML) with clay TILL - trace gravel - wet	SS	6	152	3	
			SS	7	457	3	Sieve/Hydrometer G 8% S 33% M 47% C 12%
			SS	8	381	50	
	51.8	Very dense grey silty sand (SM) with gravel TILL - wet	SS	9	483	69	Sieve G 18% S 57% Fines 25%
8.18	51.2	End of Borehole Groundwater was encountered at a depth of 6.8 m in the open borehole					



BACKFILL/MONITOR WELL/PIEZOMETER

BACKFILL SYMBOL: ASPHALT, GROUT, CONCRETE, BENTONITE, DRILL CUTTINGS, SAND, SLOUGH

Drilling Contractor: Downing	Logged By: MG
Drilling Method: HSA	Reviewed By: SS
Completion Depth: 8.18 m	Page 1 of 1

MONITORING WELL RECORD

BH24-5

CLIENT: City of Ottawa
PROJECT: 1010 Somerset
LOCATION: 1010 Somerset, Ottawa, ON
DATE BORED: 10/24/2024 to 10/25/2024

MW COORDINATES: 5028408.9N 444023.4E
WATER LEVEL: 4.2 m on 10/28/2024

PROJECT NO.: 160402067
MW ELEVATION: 59.80m
DATUM: Geodetic

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	SAMPLE TYPE	NUMBER	RECOVERY (mm) or TCR %	N-VALUE or RQD %	OTHER TESTS / REMARKS
0	59.8						
	59.7	TOPSOIL - silty sand containing rootlets and organics					
		FILL: grey sand with gravel - moist	SS	1	360	22	
			SS	2	410	32	
	58.3	FILL: brown silty sand with gravel - moist	SS	3	360	11	Sieve G 20% S 39% Fines 41%
	57.5	Very stiff grey CLAY (CH) - moist	SS	4	610	7	Sieve/Hydrometer G 0% S 10% M 40% C 50%
		* Undrained Shear Strength >118 kPa	SV	-	-	-	
		* Undrained Shear Strength >118 kPa	SV	-	-	-	
			SS	5	610	3	
			SV	-	-	-	
		* Undrained Shear Strength >118 kPa	SV	-	-	-	
	54.5	Loose to compact grey silty sand (SM) with gravel TILL - with occasional to frequent cobbles and boulders - trace clay - wet	SS	6	610	1	
			SS	7	560	7	
			SS	8	610	12	Sieve/Hydrometer G 14% S 49% M 28% C 9%
			SS	9	560	13	
	51.6	End of Borehole					

UNDRAINED SHEAR STRENGTH, Cu (kPa): LABORATORY TEST ▲ FIELD VANE TEST ◆ POCKET PEN. ★ REMOULD VANE TEST ◻ — 50 kPa, 100 kPa, 150 kPa, 200 kPa

WATER CONTENT & ATTERBERG LIMITS W_P W W_L

SPT (N-value) BLOWS/0.3m ●

Water Content (%) and Blow Count: 10 20 30 40 50 60 70 80

BACKFILL/ MONITOR WELL/ PIEZOMETER

▼ Water Level Measured On Date Indicated

BACKFILL SYMBOL: BENTONITE, ASPHALT, DRILL CUTTINGS, GROUT, SAND, CONCRETE, SLOUGH

Drilling Contractor: Downing	Logged By: MG
Drilling Method: HSA	Reviewed By: SS
Completion Depth: 8.23 m	Page 1 of 1

BOREHOLE RECORD

BH24-6

CLIENT: City of Ottawa
PROJECT: 1010 Somerset
LOCATION: 1010 Somerset, Ottawa, ON
DATE BORED: 10/24/2024

BH COORDINATES: 5028372.5N 444013.9E
WATER LEVEL: N/A

PROJECT NO.: 160402067
BH ELEVATION: 60.34m
DATUM: Geodetic

UNDRAINED SHEAR STRENGTH, Cu (kPa): LABORATORY TEST ▲, FIELD VANE TEST ◆, POCKET PEN. ★, REMOULD VANE TEST ▣; 50 kPa, 100 kPa, 150 kPa, 200 kPa
WATER CONTENT & ATTERBERG LIMITS W_P W W_L
SPT (N-value) BLOWS/0.3m ●
Water Content (%) and Blow Count: 10 20 30 40 50 60 70 80

Elevation (m)	Soil Description (USCS)	Type	Number	Recovery (mm) or TCR %	N-Value or RQD %	Other Tests / Remarks
60.3 / 60.3	ASPHALT - 75 mm					
	FILL: brown sand with gravel - moist	SS	1	50	-	50/140mm >>
59.6	FILL: brown sand with gravel - moist	SS	2	25	13	
58.8	Stiff to very stiff grey CLAY (CH) - moist - low to medium sensitivity	SS	3	610	5	
		SV	-	-	-	
		SV	-	-	-	
		SS	4	610	2	
		SV	-	-	-	
	* Undrained Shear Strength >118 kPa	SV	-	-	-	
55.8	Compact grey sandy silt (ML) with clay and gravel TILL - wet	SS	5	280	13	Sieve/Hydrometer G 13%, S 24%, M 50%, C 13%
55.2	Compact grey clayey sand (SC) with gravel TILL - moist	SS	6	305	21	
54.1	Compact grey silty sand (SM) TILL - wet	SS	7	410	29	
		SS	8	380	18	
		SS	9	430	15	
52.1	End of Borehole					

BACKFILL SYMBOL: ASPHALT, GROUT, CONCRETE, BENTONITE, DRILL CUTTINGS, SAND, SLOUGH

Drilling Contractor: Downing
Drilling Method: HSA
Completion Depth: 8.23 m
Logged By: MG
Reviewed By: SS

BOREHOLE RECORD

Stantec

BH24-7

CLIENT: **City of Ottawa**
PROJECT: **1010 Somerset**
LOCATION: **1010 Somerset, Ottawa, ON**
DATE BORED: **11/08/2024**

BH COORDINATES: 5028397.4N 444039.2E
WATER LEVEL: **N/A**

PROJECT NO.: **160402067**
BH ELEVATION: **60.13m**
DATUM: **Geodetic**

UNDRAINED SHEAR STRENGTH, Cu (kPa): LABORATORY TEST ▲; FIELD VANE TEST ◆; POCKET PEN. ★; REMOULD VANE TEST ◘; 50 kPa, 100 kPa, 150 kPa, 200 kPa

WATER CONTENT & ATTERBERG LIMITS: W_P W W_L

SPT (N-value) BLOWS/0.3m ●

Water Content (%) and Blow Count: 10 20 30 40 50 60 70 80

DEPTH (m)	ELEVATION (m)	SOIL DESCRIPTION (USCS)	TYPE	NUMBER	RECOVERY (mm) or TCR %	N-VALUE or RQD %	OTHER TESTS / REMARKS
0	60.1						
	60.1	TOPSOIL - silty sand with gravel containing rootlets and organics					
		FILL: brown clayey silty sand with gravel containing cobbles, boulders, and debris of asphalt and concrete - moist - contains cobbles and boulders - contains miscellaneous debris including asphalt and concrete	GS	1	-	-	
			GS	2	-	-	
	57.5	Stiff brown lean CLAY (CL) - weathered crust - moist	SS	3	610	10	
			ST	4	-	-	
	55.6	Soft grey lean CLAY to Clayey SILT (CL-ML) with gravel - wet	SS	5	483	2	
	54.8	Dense grey silty sand (SM) TILL - trace clay - moist	SS	6	483	35	
	54.1	Compact grey clayey sand (SC) TILL - trace to some gravel - moist to wet	SS	7	381	29	Sieve G 8% S 54% Fines 38%
			SS	8	203	23	Sieve G 18% S 50% Fines 32%
			SS	9	356	27	
	51.9	End of Borehole Groundwater encountered at a depth of 4.6 m in the open borehole					

BACKFILL SYMBOL: ASPHALT, GROUT, CONCRETE, BENTONITE, DRILL CUTTINGS, SAND, SLOUGH

Drilling Contractor: Downing
Drilling Method: HSA
Completion Depth: 8.23 m

Logged By: MG
Reviewed By: SS

DILLON CONSULTING

MW21-13

Client: City of Ottawa
Project: 1010 Somerset Street West
Project No.: 21-2419
Location: Ottawa, Ontario
Drilling Co.: Aardvark
Drilling Method: Auger/ Air Hammer
Supervised by: EB
Date Started: 2021-09-13
Date Completed: 2021-09-13

Surface Elevation (masl): 59.18
Top of casing (masl): 59.03

Stratigraphic Description	Depth (m)
Topsoil Brown, organics (roots)	0.2
Sandy gravel	1.12
Fill Sandy clay, dry, some organics (roots)	1.37
Fill Grey clay, some rocks, dry, dense, bits of cinder	2.39
Fill Red brown gravelly sand, dry, loose	2.43
Clay Grey brown clay, dry transitioning to moist at 3.8m to saturated at 4.25m, dense	4.42
Till Sandy gravelly grey clay, saturated	5.33
Till Silty clay till	5.46
TILL Sandy silt and gravel	6.1
End of borehole	

Well Construction:
Flushmount casing and solid 0.05m PVC through cement
Solid 0.05m PVC through bentonite
Slotted 0.05m PVC through sand pack

Method	Number	PID	Rec %
Split Spoon	SS2	45/8	85
Split Spoon	SS3*	65/10	58
Split Spoon	SS4	---	23
Split Spoon	SS5	45/2	79
Split Spoon	SS6*	60/6	100
Split Spoon	SS7	75/11	94
Split Spoon	SS8	---	42

m bgs - meters below ground surface
m asl - meters above sea level
* Indicates sample submitted for analysis

LITHOLOGY SYMBOLS: Organics; Fill (made ground); Glacial Till; Sandy Gravel; Clay

SAMPLE TYPE: Split Spoon

DILLON CONSULTING

MW21-15

Client: City of Ottawa
Project No.: 21-2419
Drilling Co.: Aardvark
Supervised by: EB

Project: 1010 Somerset Street West
Location: Ottawa, Ontario
Drilling Method: Auger/ Air Hammer
Date Started: 2021-09-13 Date Completed: 2021-09-14

Surface Elevation (masl): 59.80
Top of casing (masl): 59.67

Depth (m)	Stratigraphic Description	Well Construction
0.03	**Asphalt**	Flushmount casing and solid 0.05m PVC through cement
0.15	**Fill** Light grey gravelly sand, dry	
	Asphalt	
1.22	**Fill** Clayey sand, red brick pieces, some small gravel at 1.5m, compact	Solid 0.05m PVC through bentonite
2.29	**Clay** Dry, compact	
3.52	**Silty clay** Less compact	
3.81	**Silt** Grey, soft, damp transition to wet at 4.8m	Slotted 0.05m PVC through sand pack
4.98	**Sandy silt** Wet, some rocks observed at 5.3m	
6.71	End of borehole	

Sample Method	Number	PID	Rec %
Split Spoon	SS1*	5/5	73
Split Spoon	SS2	---	54
Split Spoon	SS3	---	85
Split Spoon	SS4	40/7	100
Split Spoon	SS5*	70/7	100
Split Spoon	SS6	---	100
Split Spoon	SS7	55/5	100
Split Spoon	SS8	50/0	75

m bgs - meters below ground surface
m asl - meters above sea level
* Indicates sample submitted for analysis

LITHOLOGY SYMBOLS: Asphalt, Clay, Silt, Fill (made ground), Silt / Clay, Sandy Silt

SAMPLE TYPE: Split Spoon

DILLON CONSULTING

MW21-20D

Client: City of Ottawa	Project: 1010 Somerset Street West
Project No.: 21-2419	Location: Ottawa, Ontario
Drilling Co.: Aardvark	Drilling Method: Auger/ Air Hammer
Supervised by: EB	Date Started: 2021-09-15 Date Completed: 2021-09-15

Surface Elevation (masl): 60.04

Top of casing (masl): 59.94

Depth (m)	Stratigraphic Description
0.08 / 0.15 / 0.32	**Topsoil** Organics (roots)
	Fill Dark brown clay fill
1.57 / 1.68	**Fill** sand and gravel fill
	Fill Clay fill, red brick fragments, cinder, orange mottling
5.33	**Fill** Sand fill
	Clay Transition from compact and dry to damp, less compact at 3.8m, to saturated at 4.5m
7.62	**Till** Grey gravelly sand, saturated, coarse gravel, mica, and feldspar pieces observed 6.8m to 7.6m
	Bedrock - Limestone Grey
30.48	End of borehole

Well Construction

- Flushmount casing and solid 0.05m PVC through cement
- Solid 0.05m PVC through bentonite
- Slotted 0.05m PVC through sand pack
- Back filled with holeplug

Sample

Method	Number	PID	Rec %
Split Spoon	SS1	---	52
Split Spoon	SS2*	---	63
Split Spoon	SS3	95/1	100
Split Spoon	SS4*	25/2	100
Split Spoon	SS5	25/4	100
Split Spoon	SS6* DUP5*	55/7	100
Air Rotary Drill			

Depth Scale (m): 1.0 – 30.0

m bgs - meters below ground surface
m asl - meters above sea level
* Indicates sample submitted for analysis

LITHOLOGY SYMBOLS: Organics; Clay; Bedrock; Fill (made ground); Glacial Till

SAMPLE TYPE: Split Spoon; Air Rotary Drill

PATERSON GROUP

Photograph of Rock Cores – BH1-25 – RC1 & RC2

Photograph of Rock Core obtained from BH 1-25 from interval RC1 and RC2

RC1 Rock Core interval ranged between 28’2” to 29’10”

Recovery (%) = 100

Rock Quality Designation (RQD) = 80

RC2 Rock Core interval ranged between 29’10” to 32’11”

Recovery (%) = 100

Rock Quality Designation (RQD) = 100

Photograph of Rock Cores – BH2-25 – RC1 & RC2

Photograph of Rock Core obtained from BH 2-25 from interval RC1 and RC2

RC1 Rock Core interval ranged between 26’8” to 29’8”

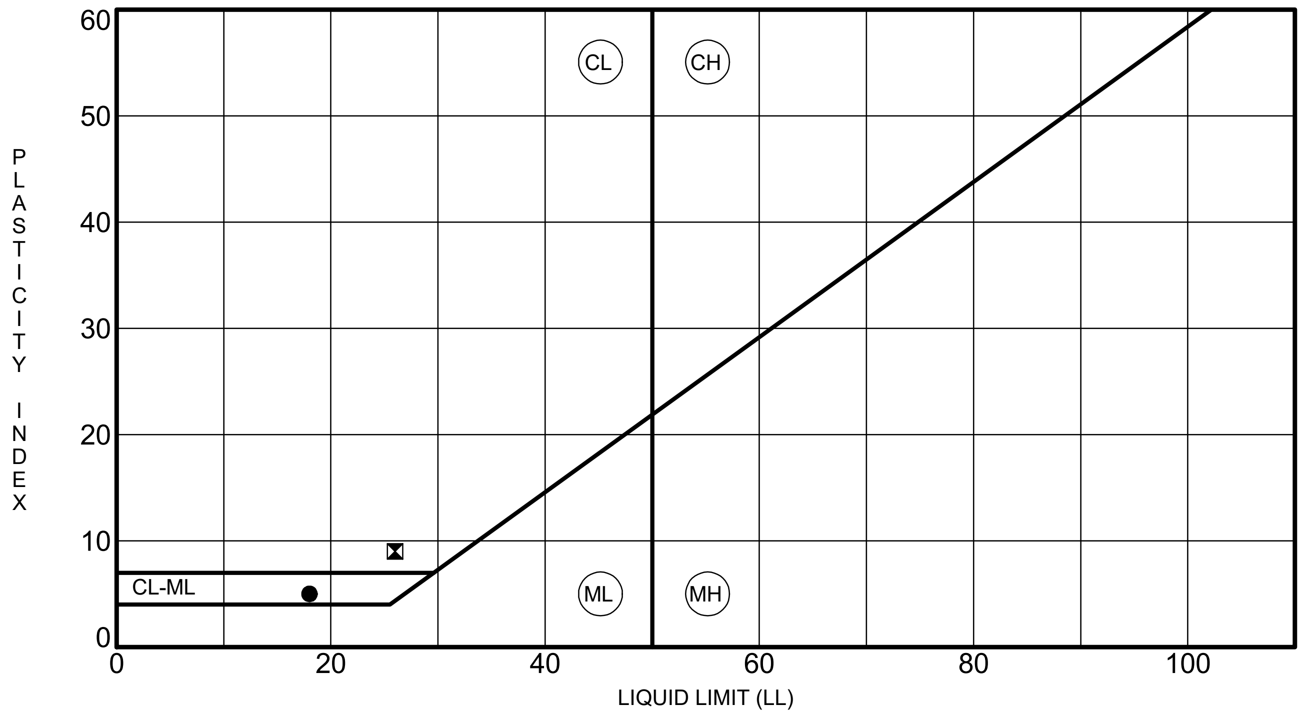
Recovery (%) = 100

Rock Quality Designation (RQD) = 77

RC2 Rock Core interval ranged between 29’8” to 32’10”

Recovery (%) = 100

Rock Quality Designation (RQD) = 97



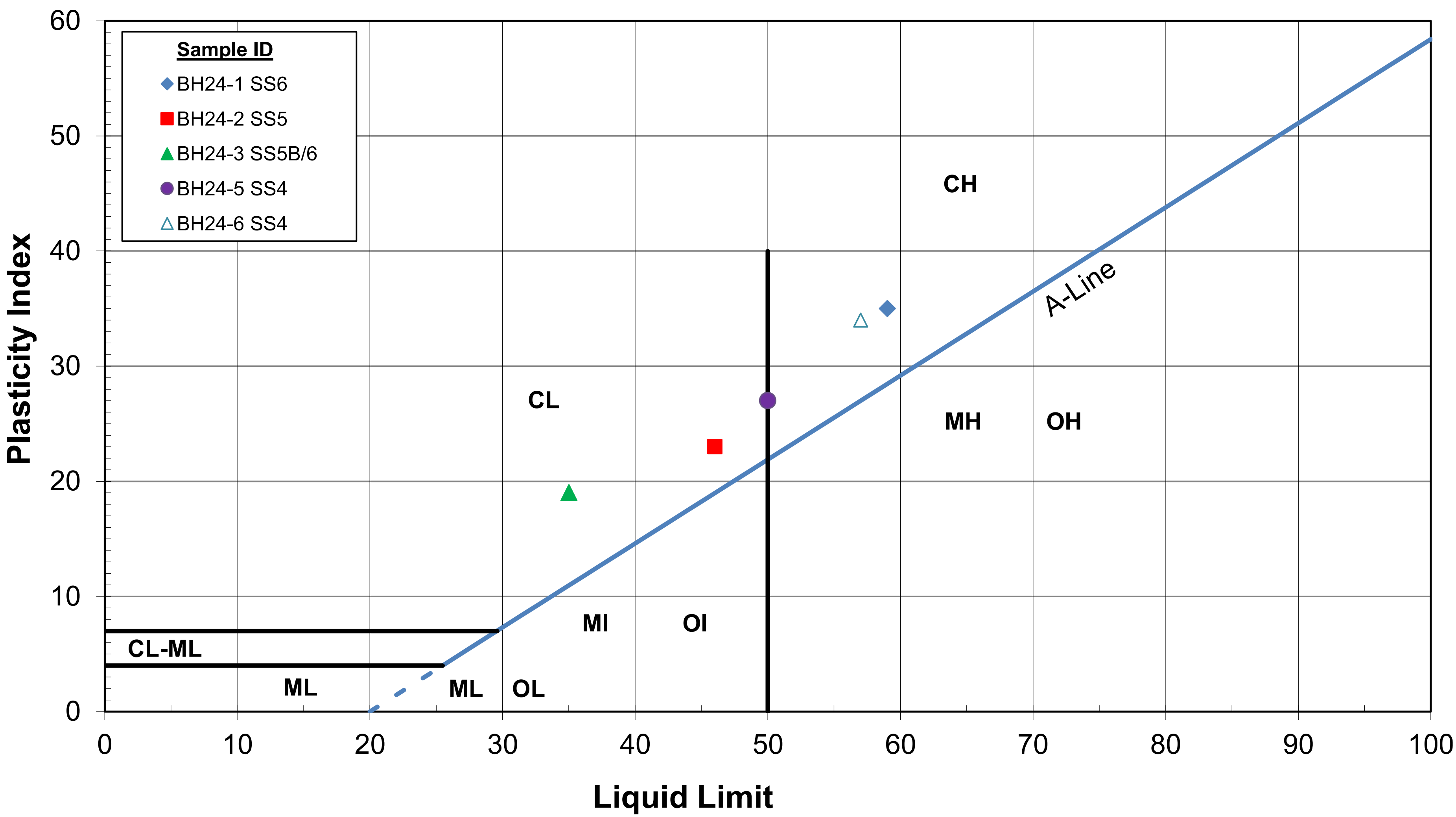
	Specimen Identification		LL	PL	PI	Fines	Classification
●	**BH 1-25**	**SS6**	**18**	**13**	**5**		**CL-ML**
⊠	**BH 3-25**	**SS7**	**26**	**17**	**9**		**CL - Inorganic clays with low plasticity**

CLIENT	**Conseil des ecoles publiques de l'Est de l'Ontario**	FILE NO.	**PG7468**
PROJECT	**Geotechnical Investigation -**	DATE	**19 Mar 25**
	1010 Somerset Street West		

PATERSON GROUP

9 Auriga Drive
Ottawa, Ontario
K2E 7T9
TEL: (613) 226-7381

ATTERBERG LIMITS' RESULTS



Stantec

Silty Clay (CL-CH)

1010 Somerset West Ottawa

PLASTICITY CHART

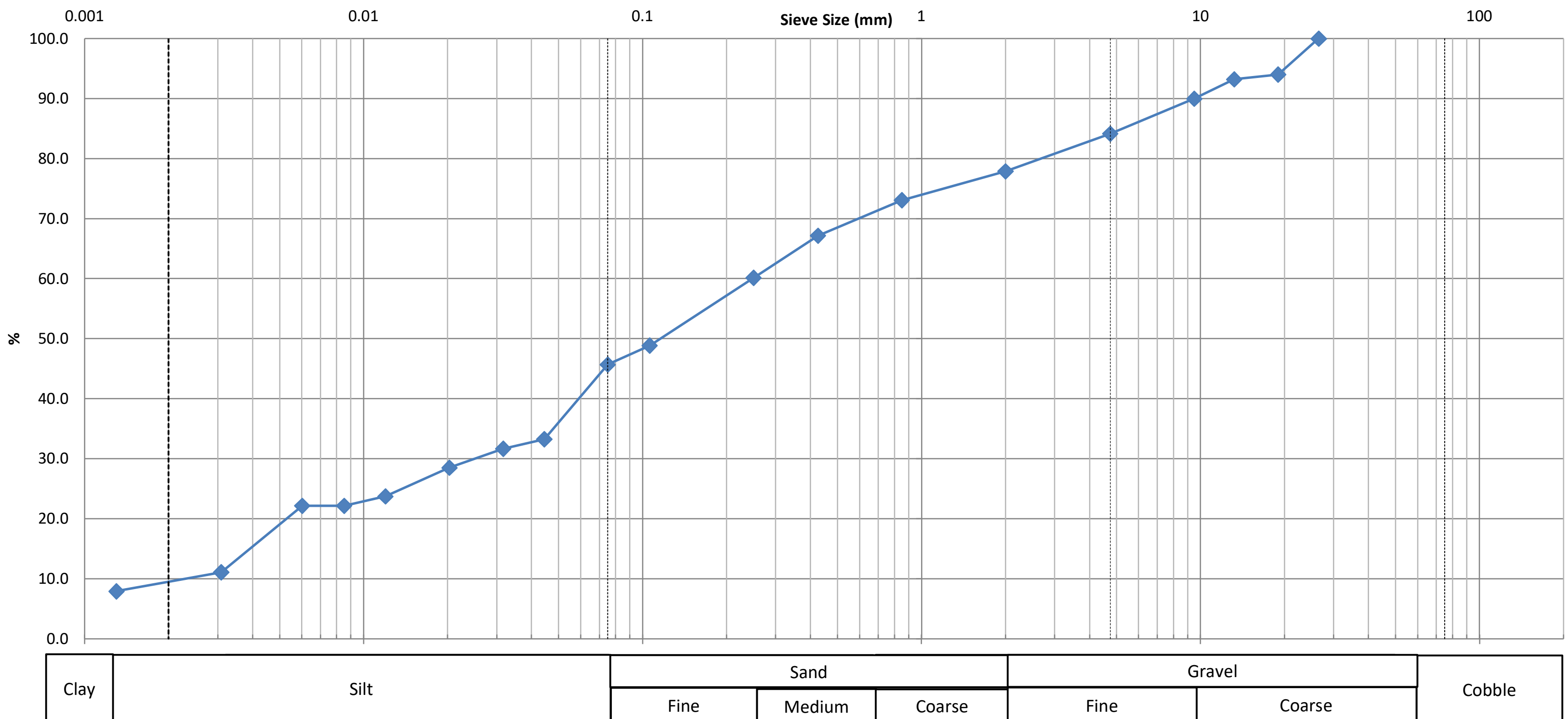
Figure No. E3

Project No. 160402067

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	CEPEO	DEPTH:	-	FILE NO:	PG7468
CONTRACT NO.:		BH OR TP No.:	BH1-25 SS7	LAB NO:	59063
PROJECT:	1010 Somerset St W			DATE RECEIVED:	14-Mar-25
				DATE TESTED:	17-Mar-25
DATE SAMPLED:	-			DATE REPORTED:	21-Mar-25
SAMPLED BY:	NV			TESTED BY:	C.M



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					12.5%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					15.8	38.4	36.2	9.5		

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	CEPEO	DEPTH:	-	FILE NO.:	PG7468
PROJECT:	1010 Somerset St W	BH OR TP No.:	BH1-25 SS7	DATE SAMPLED:	-
LAB No. :	59063	TESTED BY:	C.M	DATE RECEIVED:	14-Mar-25
SAMPLED BY:	NV	DATE REPT'D:	21-Mar-25	DATE TESTED:	17-Mar-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
760.9	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	48.66	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	20.96	AIR DRY	781.90	781.90
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	760.90	760.90
		CORRECTED	0.973	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5	**0.0**	0.0	**100.0**
19	**45.4**	6.0	**94.0**
13.2	**51.6**	6.8	**93.2**
9.5	**76.0**	10.0	**90.0**
4.75	**120.6**	15.8	**84.2**
2.0	**168.3**	22.1	**77.9**
Pan	**592.6**		
0.850	**3.09**	26.9	**73.1**
0.425	**6.89**	32.9	**67.1**
0.250	**11.38**	39.8	**60.2**
0.106	**18.62**	51.1	**48.9**
0.075	**20.65**	54.3	**45.7**
Pan	**20.96**		
SIEVE CHECK 0.0	MAX = 0.3%		

HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^oC)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:19	27.0	6.0	23.0	0.0445	42.7	**33.2**
2	8:20	26.0	6.0	23.0	0.0317	40.6	**31.7**
5	8:23	24.0	6.0	23.0	0.0203	36.6	**28.5**
15	8:33	21.0	6.0	23.0	0.0120	30.5	**23.7**
30	8:48	20.0	6.0	23.0	0.0085	28.5	**22.2**
60	9:18	20.0	6.0	23.0	0.0060	28.5	**22.2**
250	12:28	13.0	6.0	23.0	0.0031	14.2	**11.1**
1440	8:18	11.0	6.0	23.0	0.0013	10.2	**7.9**

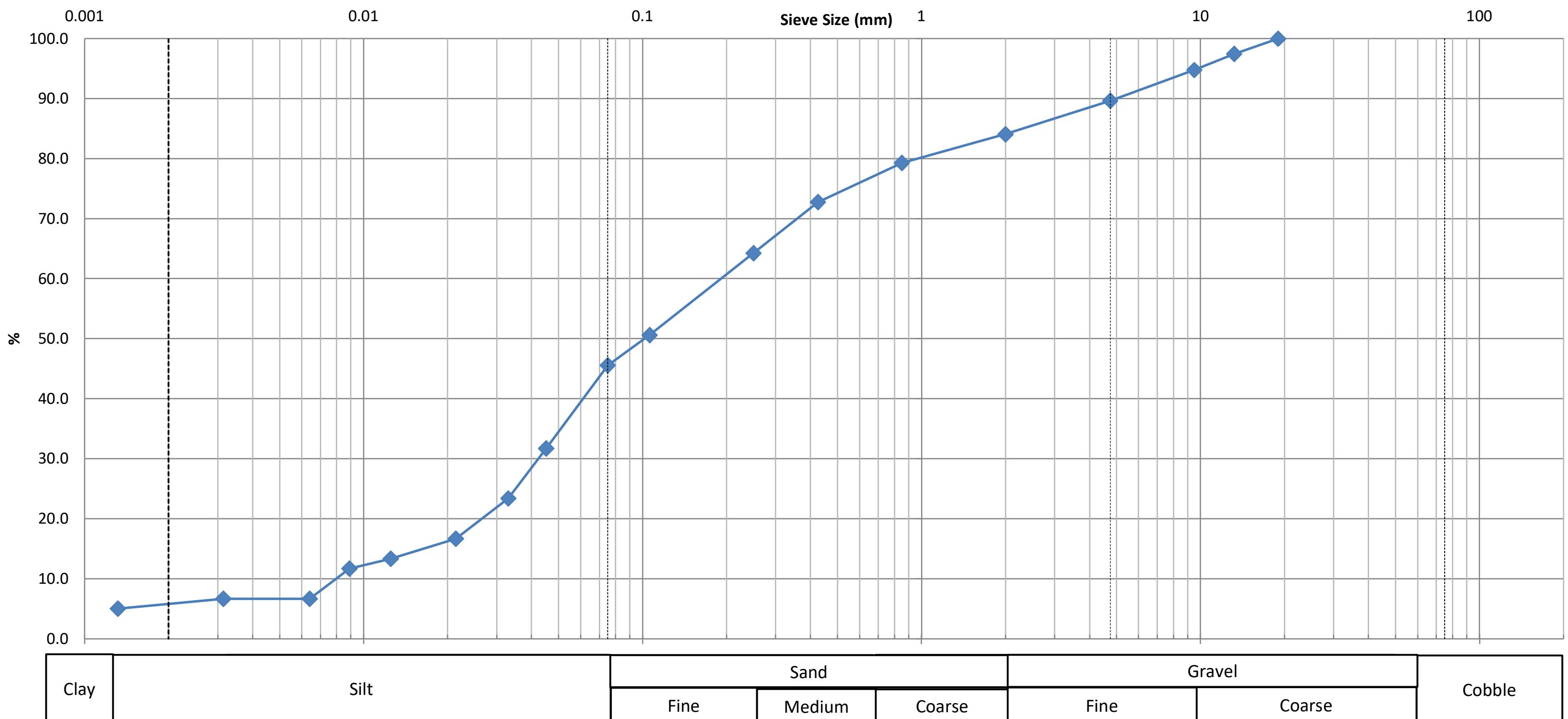
Moisture = 12.5%

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

PATERSON GROUP

SIEVE ANALYSIS
ASTM C136

CLIENT:	CEPEO	DEPTH:	-	FILE NO:	PG7468
CONTRACT NO.:		BH OR TP No.:	BH2-25 SS9	LAB NO:	59064
PROJECT:	1010 Somerset St W			DATE RECEIVED:	14-Mar-25
				DATE TESTED:	17-Mar-25
DATE SAMPLED:	-			DATE REPORTED:	21-Mar-25
SAMPLED BY:	NV			TESTED BY:	C.M



Identification	Soil Classification				MC(%)	LL	PL	PI	Cc	Cu
					10.6%					
	D100	D60	D30	D10	Gravel (%)	Sand (%)	Silt (%)	Clay (%)		
					10.3	44.1	39.9	5.6		

Comments:

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

PATERSON GROUP

HYDROMETER
LS-702 ASTM-422

CLIENT:	CEPEO	DEPTH:	-	FILE NO.:	PG7468
PROJECT:	1010 Somerset St W	BH OR TP No.:	BH2-25 SS9	DATE SAMPLED:	-
LAB No. :	59064	TESTED BY:	C.M	DATE RECEIVED:	14-Mar-25
SAMPLED BY:	NV	DATE REPT'D:	21-Mar-25	DATE TESTED:	17-Mar-25

SAMPLE INFORMATION

SAMPLE MASS	SPECIFIC GRAVITY
617.6	2.700

		HYGROSCOPIC MOISTURE		
INITIAL WEIGHT	50.00			
WEIGHT CORRECTED	49.81	TARE WEIGHT	0.00	ACTUAL WEIGHT
WT. AFTER WASH BACK SIEVE	23.37	AIR DRY	619.90	619.90
SOLUTION CONCENTRATION	40 g/L	OVEN DRY	617.60	617.60
		CORRECTED	0.996	

GRAIN SIZE ANALYSIS

SIEVE DIAMETER (mm)	WEIGHT RETAINED (g)	PERCENT RETAINED	PERCENT PASSING
26.5			
19	**0.0**	0.0	**100.0**
13.2	**15.7**	2.5	**97.5**
9.5	**32.3**	5.2	**94.8**
4.75	**63.9**	10.3	**89.7**
2.0	**98.3**	15.9	**84.1**
Pan	**519.3**		
0.850	**2.84**	20.7	**79.3**
0.425	**6.73**	27.2	**72.8**
0.250	**11.78**	35.7	**64.3**
0.106	**19.89**	49.4	**50.6**
0.075	**22.90**	54.4	**45.6**
Pan	**23.37**		
SIEVE CHECK 0.0	MAX = 0.3%		

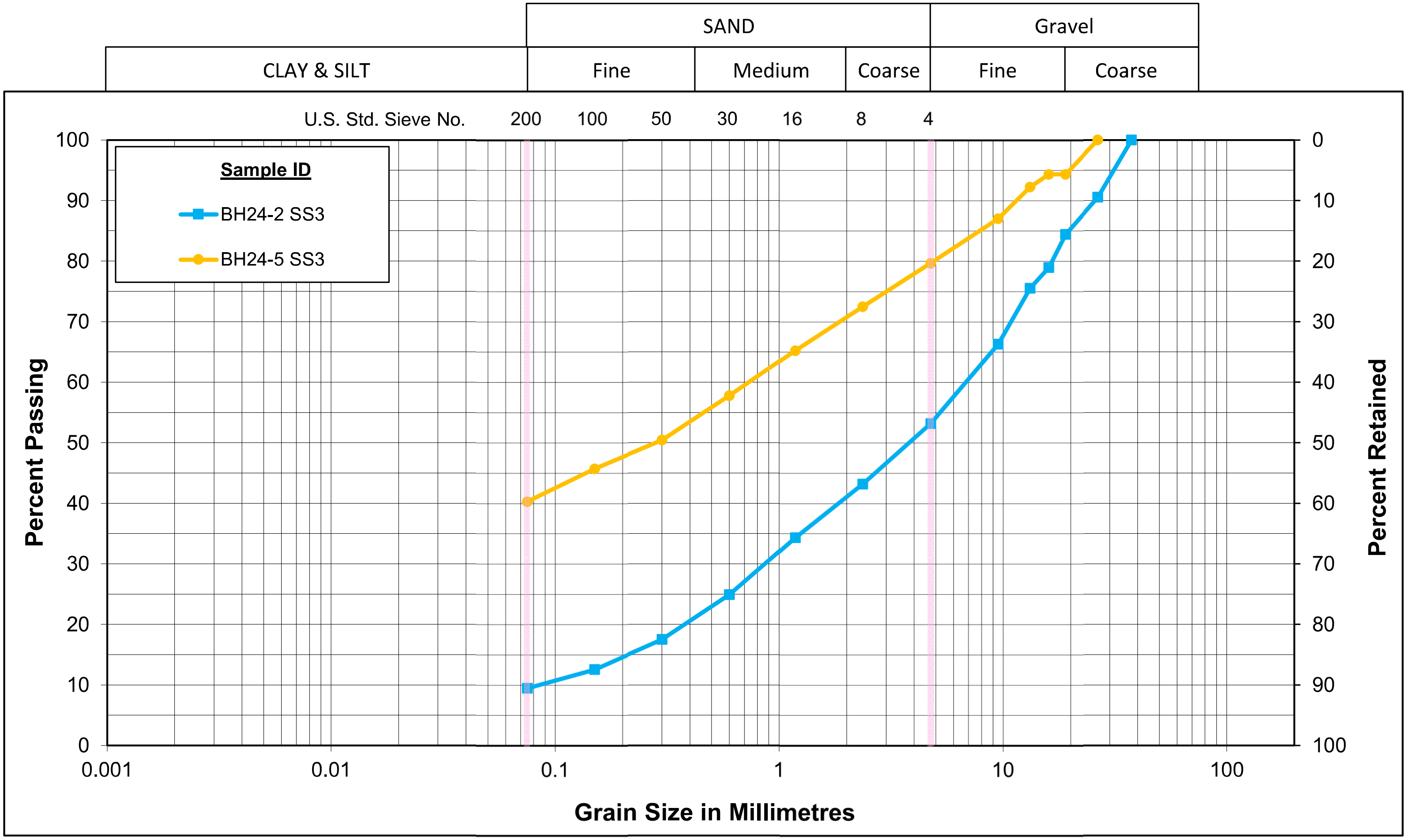
HYDROMETER DATA

ELAPSED	TIME (24 hours)	Hs	Hc	Temp. (^{o}C)	DIAMETER	(P)	TOTAL PERCENT PASSING
1	8:02	25.0	6.0	23.0	0.0451	37.7	**31.7**
2	8:03	20.0	6.0	23.0	0.0330	27.8	**23.4**
5	8:06	16.0	6.0	23.0	0.0214	19.8	**16.7**
15	8:16	14.0	6.0	23.0	0.0125	15.9	**13.4**
30	8:31	13.0	6.0	23.0	0.0089	13.9	**11.7**
60	9:01	10.0	6.0	23.0	0.0064	7.9	**6.7**
250	12:11	10.0	6.0	23.0	0.0031	7.9	**6.7**
1440	8:01	9.0	6.0	23.0	0.0013	6.0	**5.0**

Moisture = 10.6%

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

Unified Soil Classification System



GRAIN SIZE DISTRIBUTION

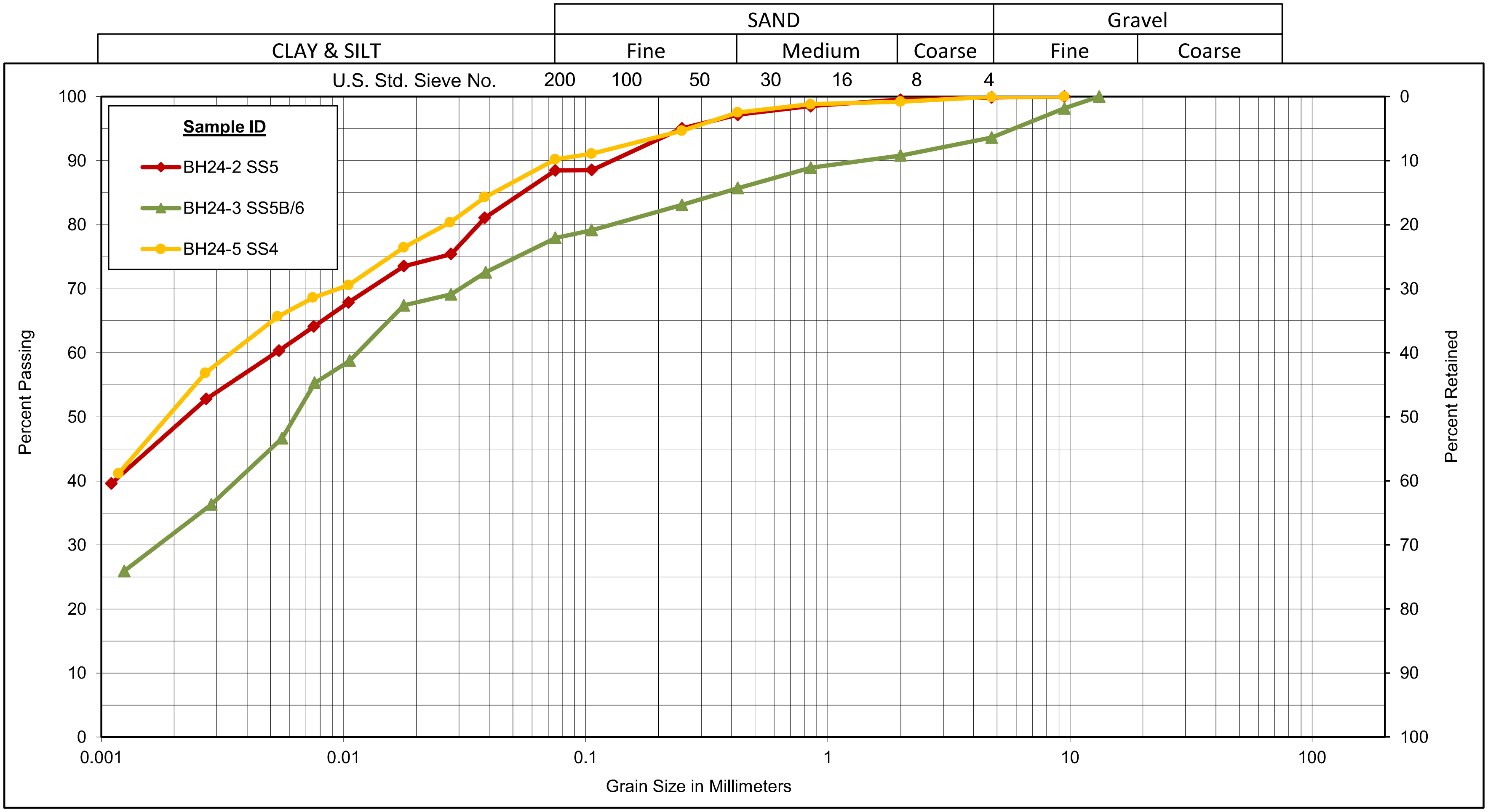
Fill Material

1010 Somerset West Ottawa

Figure No. E1

Project No. 160402067

Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH24-2 SS5	12'6"-14'6"	0.1	11.4	41.5	47.0
BH24-3 SS5B/6	8'6"-14'6"	6.4	15.7	46.9	31.0
BH24-5 SS4	7'6"-9'6"	0.1	9.7	40.2	50.0

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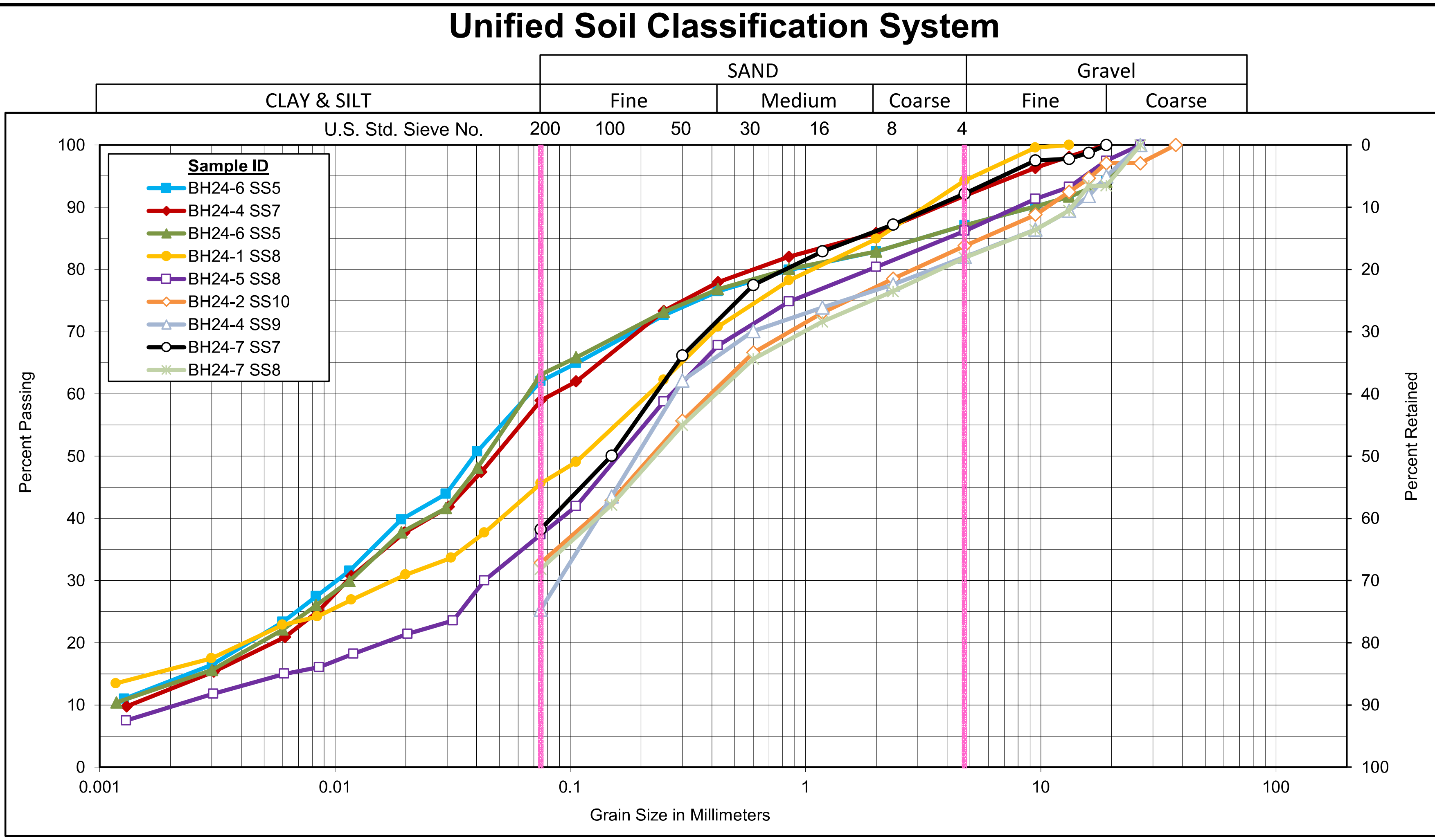
GRAIN SIZE DISTRIBUTION

Silty Clay (CL)

1010 Somerset West Ottawa

Figure No. E2

Project No. 160402067



Stantec

GRAIN SIZE DISTRIBUTION

Glacial Till (SM-ML)

1010 Somerset West Ottawa

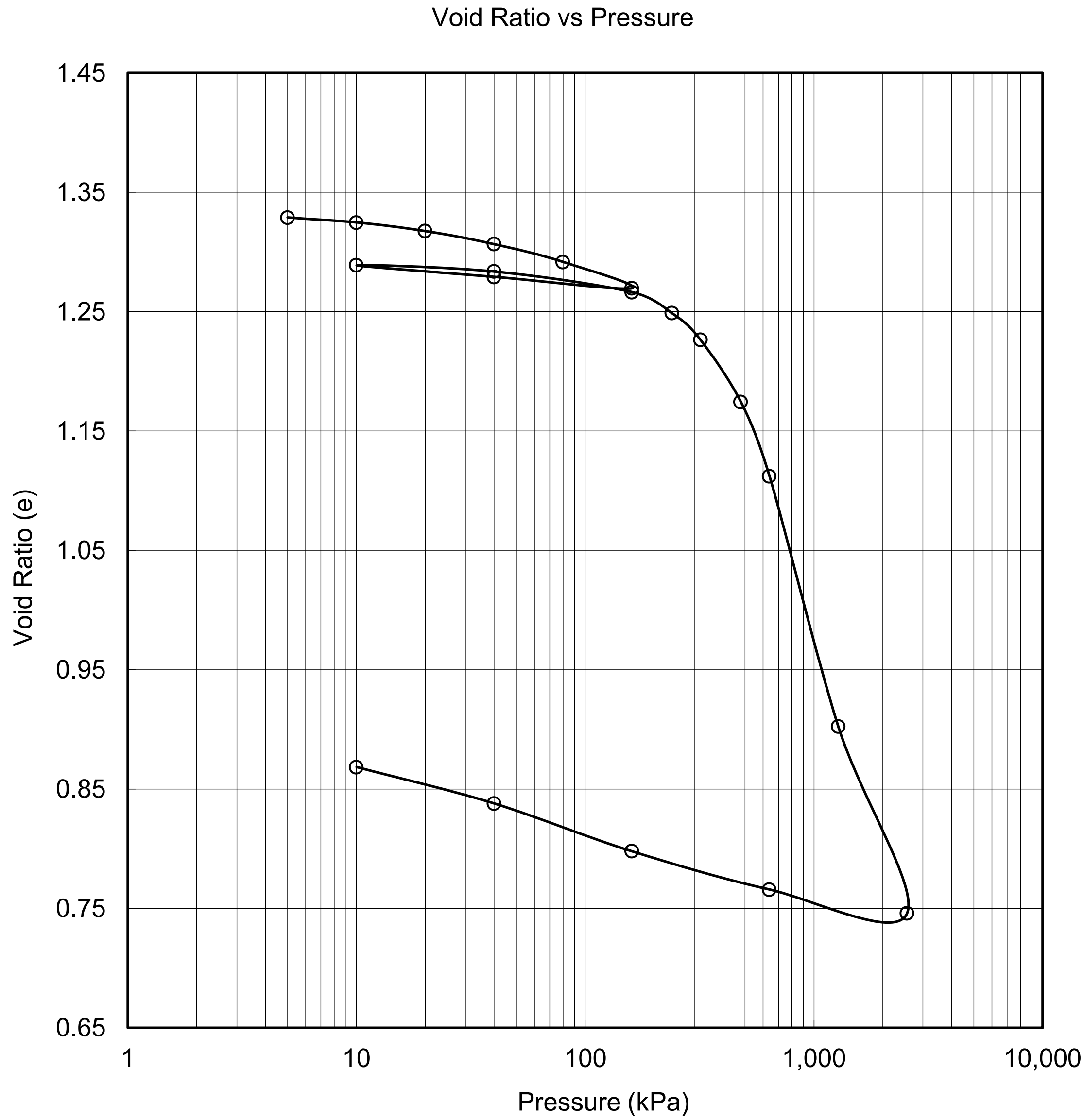
Figure No. E4

Project No. 160402067

CONSOLIDATION TEST	FIGURE E7

1010 Somerset West, Ottawa

BH 24-07, ST4



Soil Type : *Fat clay, brown, fissured, moist, CH*

e_o =	1.334	w_L =	69.7%	σ_{v0}' =	70 kPa
w =	45.2%	w_P =	25.1%	σ_P' =	400.0 kPa
γ =	17.1 kN/m^3	PI =	44.6%	C_c =	1.146
Gs =	2.805			C_r =	0.020

Project No. : 160402067.600
Date : 29-Jan-25

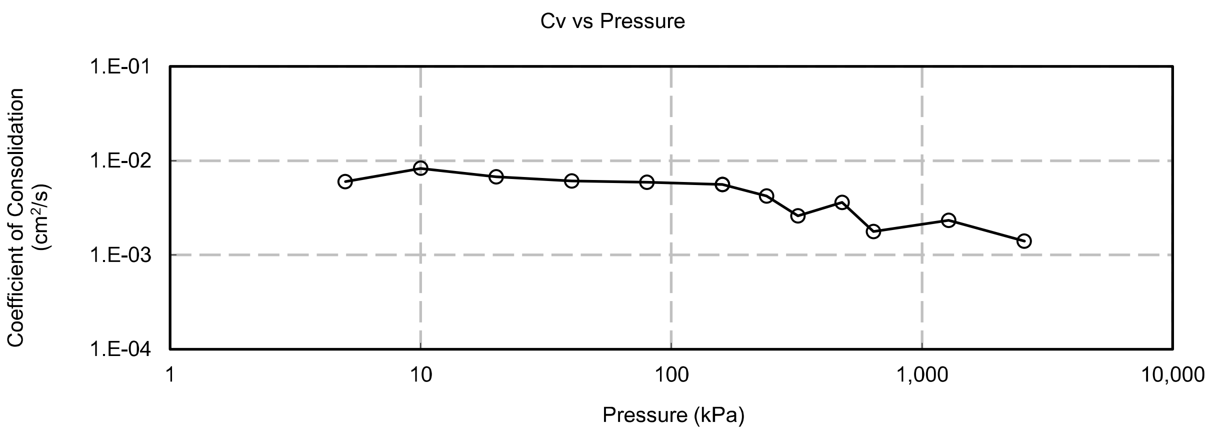
Stantec

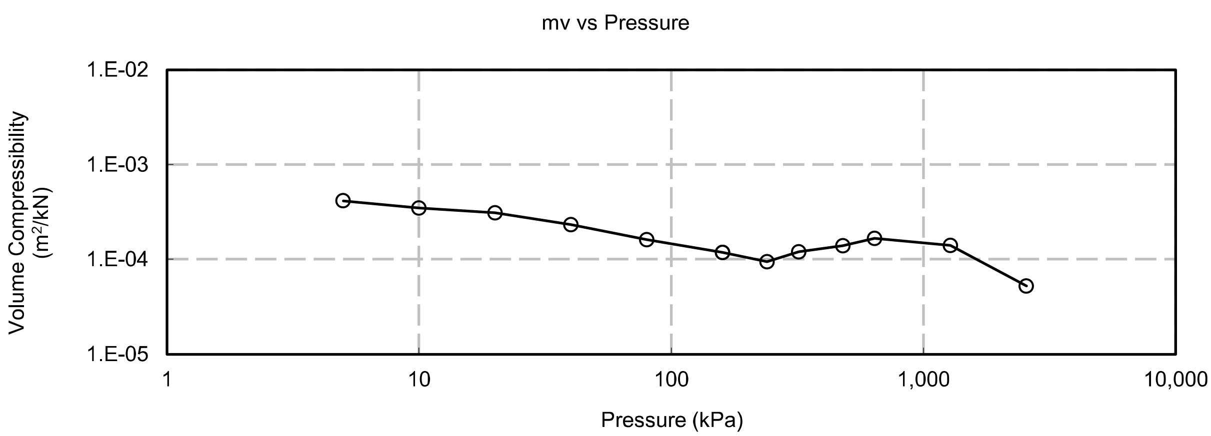
Prepared By : DB
Checked By : RG

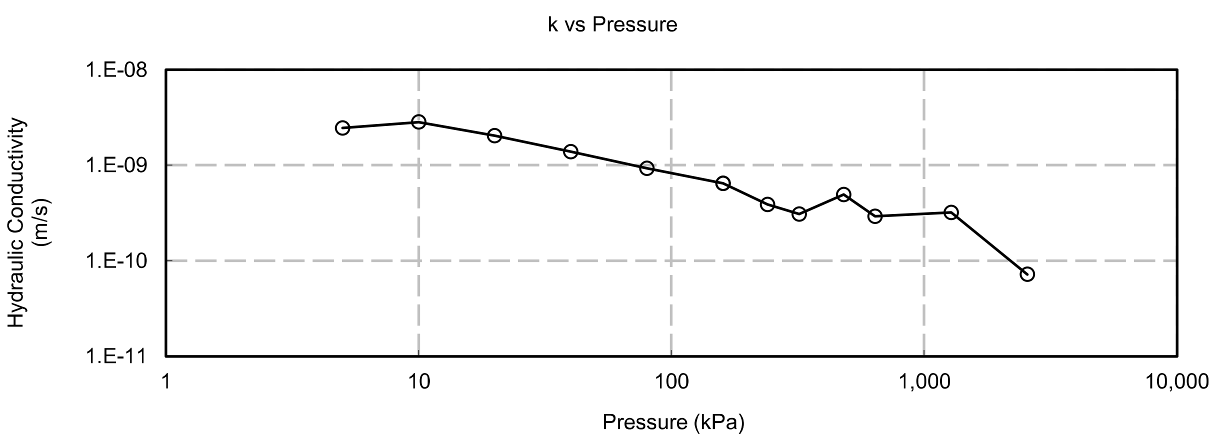
CONSOLIDATION TEST	FIGURES E8

1010 Somerset West, Ottawa

BH 24-07, ST4







Project No. :	160402067.600	Prepared By :	DB
Date :	29-Jan-25	Checked By :	RG

Certificate of Analysis

Report Date: 14-Nov-2024

Client: **Stantec Consulting Ltd. (Ottawa)**

Order Date: 11-Nov-2024

Client PO: 1010 Somerset

Project Description: 160402067.200

	MDL/Units						
Client ID:		BH24-02, SS7. 22'6"-24'6"	BH24-03, SS7. 17'6"-19'6"	-	-		
Sample Date:		25-Oct-24 09:00	24-Oct-24 09:00	-	-	-	-
Sample ID:		2446075-01	2446075-02	-	-		
Matrix:		Soil	Soil	-	-		
Physical Characteristics							
% Solids	0.1 % by Wt.	67.9	99.7	-	-	-	-
General Inorganics							
pH	0.05 pH Units	7.95	7.80	-	-	-	-
Resistivity	0.1 Ohm.m	24.9	17.8	-	-	-	-
Anions							
Chloride	10 ug/g	32	159	-	-	-	-
Sulphate	10 ug/g	152	219	-	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG7468-1 - TEST HOLE LOCATION PLAN

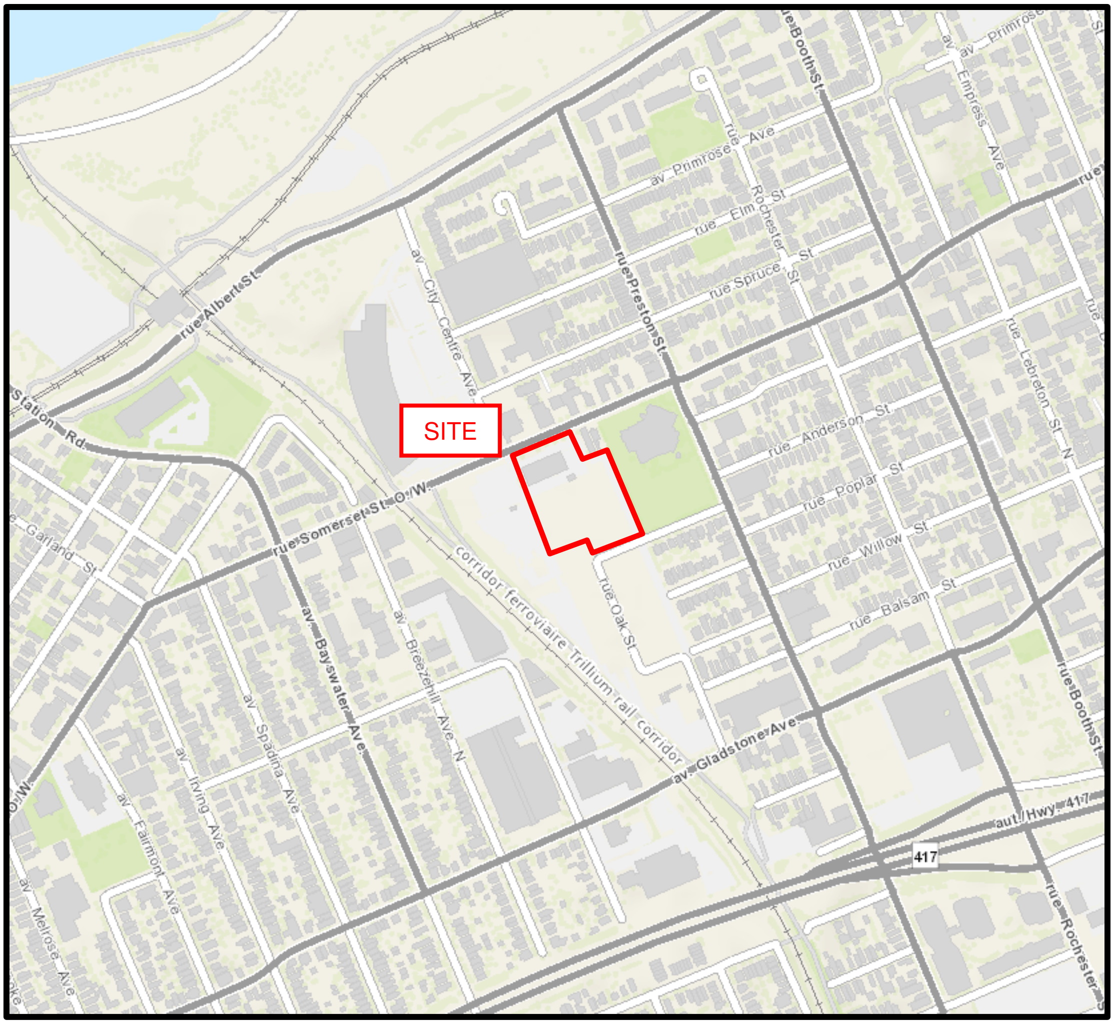


FIGURE 1

KEY PLAN

PATERSON GROUP

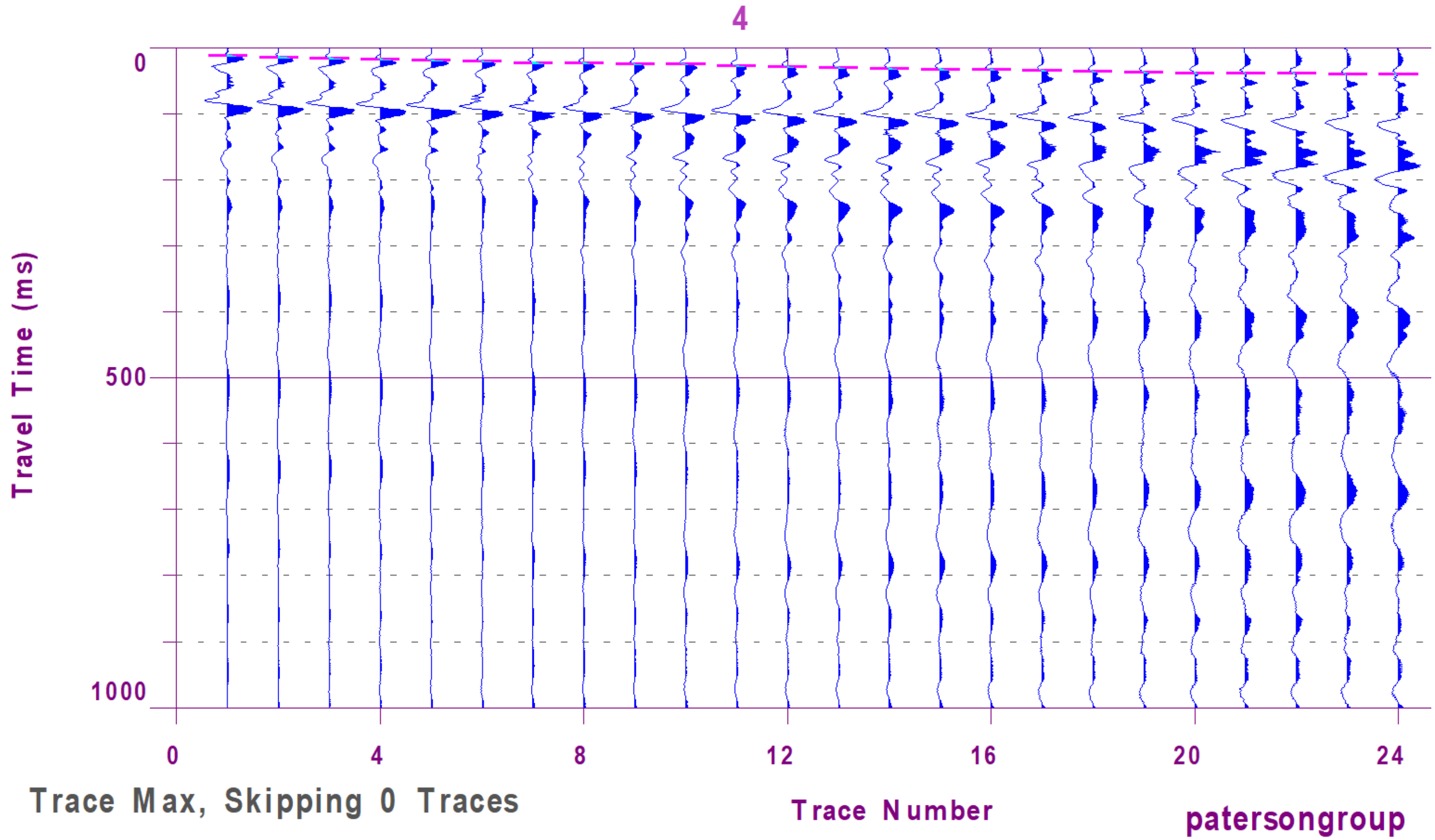


Figure 2 – Shear Wave Velocity Profile at Shot Location -14 m

PATERSON GROUP

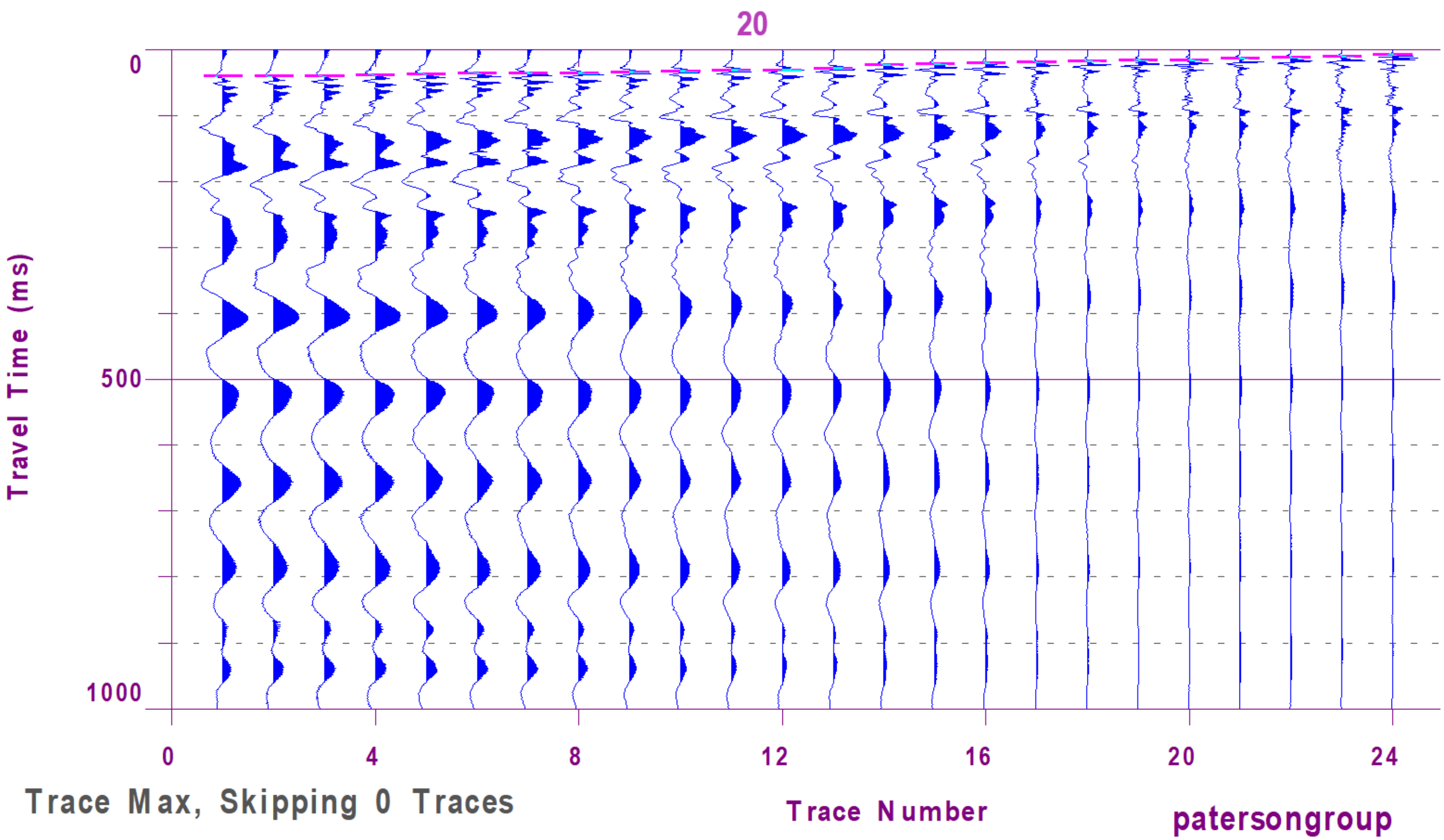
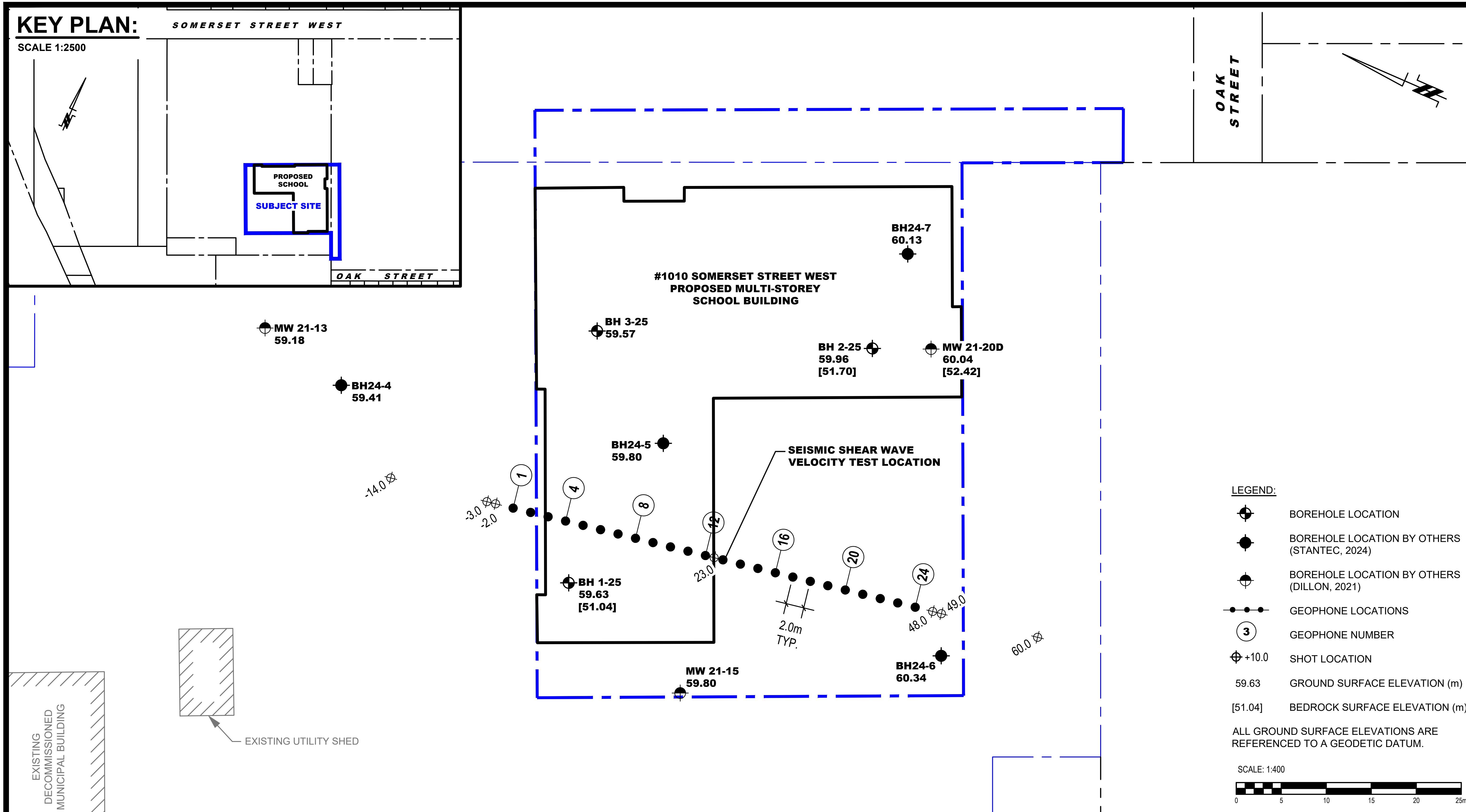


Figure 3 – Shear Wave Velocity Profile at Shot Location 60 m

PATERSON GROUP



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

NO.	REVISIONS	DATE	INITIAL

CEPEO
GEOTECHNICAL INVESTIGATION
PROPOSED SCHOOL DEVELOPMENT
1010 SOMERSET STREET WEST
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 03/2025
Drawn by: NFRV	Report No.: PG7468-1
Checked by: NFRV	Dwg. No.: PG7468-1
Approved by: DP	Revision No.:

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