Geotechnical Investigation Report
Proposed Residential Development
85 Gemini Way (Lot B)
Ottawa, Ontario

Client:

Centurion Appelt (1 Centrepoint) LP
218-3477 Lakeshore Road
Kelowna, British Columbia

Type of Document:

Final Report No. 2 (supersedes final report dated March 3,2025)

Project Number:

OTT-24014796-A0

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, Ontario K2B 8H6
t: +1.613.688.1899

Date Submitted:

November 6, 2025

Contents

List of Tables

List of Figures

List of Appendices

Executive Summary

Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed residential development to be located at 85 Gemini Way (Lot B), Ottawa, Ontario (Figure 1). Authorization to proceed with this geotechnical investigation was provided by Centurion Appelt (1 Centrepoint) LP.

Proposed Development

The proposed development will comprise of a six (6) storey wood framed building with two (2) level underground parking garage. The Architectural Drawing No. A4-2 titled, Building Section, dated September 15,2025 and prepared by Brouwer Architecture Inc. indicates that the proposed building may be supported by a raft (mat) foundation with design elevation of the lowest parking garage floor slab (the second level of the parking garage) at Elevation 79.70 m and top of raft foundation at Elevation 79.275 m. It is our understanding that consideration is also given to supporting the proposed building by footings.

A portion of the east side of the proposed new building will be located in close proximity to the west side of the neighboring existing building at 2140 Baseline Road. From the set of civil drawings, the site grading plan, Drawing No. C301, dated October 22,2025 (Revision No. 2) and prepared by LRL Engineering indicates there will be no grade raise at the site of the proposed building. The final grades will match the existing grades with some cut areas where the final grade will be lower than the existing grade.

Fieldwork Program

The borehole fieldwork was undertaken in two (2) stages. The first stage was conducted on December 12 and 19, 2024 and consists of three (3) boreholes (Borehole Nos. 24-01 to 24-03) extending to auger refusal and termination depths of 12.3 m to 15.8 m below existing grade. The second stage was undertaken on October 15 and 16, 2025 and consists of three (3) boreholes (Borehole Nos. 25-01 to 25-03) extending to cone refusal and termination depths of 9.8 m and 12.6 m depths below existing grade. Nineteen (19) mm diameter standpipe and fifty (50) mm diameter monitoring wells with screened sections were installed in selected boreholes for long-term monitoring of the groundwater levels.

A seismic shear wave velocity sounding survey was conducted at the site on September 25,2025 by Geophysics GPR International Inc. (GPR). The GPR seismic shear wave velocity sounding survey report is shown in Appendix A in the attached geotechnical report. As indicated in the GPR report, the survey was undertaken using the multi-channel analysis of surface waves (MASW), spatial auto correlation (SPAC) and seismic refraction methods.

Subsurface Conditions

The subsurface conditions consist of fill underlain by a deep sensitive marine silty clay, glacial till and limestone bedrock contacted at 13.2 m depth (Elevation 73.2 m). Based on a review of the groundwater level measurements from October 30,2025, the groundwater level is at 4.3 m and 7.3 m depths (Elevation 82.0 m and Elevation 79.0 m).

Geotechnical Engineering Comments and Recommendations

The results of the survey indicate that the average seismic shear wave velocity (V_{s30}) over a 30 m depth is 388 m/s. Based on a comparison of V_{s30} equal to 388 m/s with Sentence (2) in Section 4.1.8.4 of the 2024 Ontario Building Code (OBC), the site designation for seismic design is X_{388} which also corresponds to a Site Class C.

The subsurface soils are not considered to be susceptible to liquefaction during a seismic event.

The proposed building may be supported by conventional strip and spread footings. For a top of footing designed at the same elevation as the top of the raft elevation noted above, Elevation 79.275 m, and assuming the footing thickness is 300 mm thick, the design elevation of the underside of the footings will be at Elevation 78.975 m. Based on the borehole information, at Elevation 78.975 m, the footings will be founded on the grey silty clay. Strip and spread footings founded on the grey silty clay at Elevation 78.975 m may be designed for a bearing pressure at serviceability limit state (SLS) of 120 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 180 kPa.

The proposed building may be supported by a raft (mat) foundation designed as a drained structure with permanent perimeter and underfloor drainage systems. For a top of the raft foundation designed at Elevation 79.275 m, and assuming the raft foundation is 600 mm thick, the design elevation of the underside of the raft foundation will be at Elevation 78.675 m. Based on the borehole information, at Elevation 78.675 m, the raft foundation will be founded on the grey silty clay. A raft foundation founded on the grey silty clay at Elevation 78.675 m may be designed for a bearing pressure at serviceability limit state (SLS) of 145 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 215 kPa.

The factored ULS value includes a geotechnical resistance value of 0.5. The SLS and factored ULS values are valid provided there is no grade raise on the site. The settlements of the proposed footings or raft foundation designed for the above SLS values and properly constructed are expected to be within the normally tolerable limits of 25 mm total settlement and 19 mm differential settlement. Should the grading plan change from no site grade raise to a site grade raise or if the design elevation of the footings or raft foundation will be different from the elevation noted above, EXP should be contacted to review the acceptability of the new site grade raise and based on the approved new site grade raise provide updated bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance values for the footings or raft foundation of the proposed building.

Since a portion of the walls along the east side of the proposed new building will be located in close proximity to the foundation wall of the adjacent existing building and retaining wall (along the parking garage entrance ramp) located next door at 2140 Baseline Road and the foundations will be set at a higher elevation than the footings of the existing building at 2140 Baseline Road, the higher foundations of the new building will exert a load onto the foundation walls and a portion of the retaining wall (along the parking garage entrance ramp) of the existing building. To prevent imposed additional loads on the foundation and retaining wall of the existing building, it is recommended that the portion of the east wall of the proposed new building be supported by micropiles socketed into the underlying bedrock. The bedrock was contacted in Borehole No. 24-03 at a 13.2 m depth (Elevation 73.2 m).

Since a portion of the proposed building will be supported by a combination of either footings or a raft foundation and a portion of the east wall of the building will be supported by micro-piles, control joints may be required to prevent differential settlement. The need for a joint, type and location of the joint required and deign details regarding the joint should be provided by the structural engineer.

The lowest floor of the parking garage may be designed and constructed as a slab-on-grade placed on a 200 mm thick well-packed 19 mm sized clear stone bed placed on a minimum 300 mm thick engineered fill pad set on the approved silty clay. The engineered fill pad should consist of Ontario Provincial Standard Specification (OPSS) Granular B Type II material compacted to 98 percent standard Proctor maximum dry density (SPMDD). The clear stone will minimize the capillary rise of moisture from the sub-soil to the floor slab. Alternatively, the clear stone layer may be replaced with a 200 mm thick bed of OPSS Granular A compacted to 100 percent SPMDD overlain by a vapour barrier. Adequate saw cuts should be provided in the floor slabs to control cracking.

The excavations for the proposed building and installation of underground municipal services are anticipated to extend through the fill and into the silty clay to an 8.0 m depth below existing grade and are anticipated to be below the groundwater level.

The excavations may be undertaken by conventional heavy equipment.

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation. Within zones of seepage, the excavation side slopes are expected to slough and eventually stabilize at 2H:1V to 3H:1V from the bottom of the excavation.

If side slopes cannot be achieved due to space restrictions on site due to the proximity of open cut excavations to the property limits, existing infrastructure or to foundations of the adjacent existing building at 2140 Baseline Road, the new building construction would have to be undertaken within the confines of an engineered support system (shoring system). A conventional shoring system may consist of a soldier pile and timber lagging system or interlocking steel sheeting system.

Seepage of the surface and subsurface water into unshored and shored excavations is anticipated. However, it should be possible to remove any water entering the excavations by collecting it in perimeter ditches or low points and pumping from sumps. In areas of high infiltration or in areas where more permeable soil layers may exist, such as the silty sand layers in the silty clay, a higher seepage rate should be anticipated and will require high-capacity pumps to keep the excavation dry.

The materials that will be excavated include asphaltic concrete, fill, silty clay, glacial till and limestone bedrock. These materials are not considered suitable for re-use as backfill material for the proposed new building and should be discarded. There fore, it is anticipated that the majority of the fill required for use as backfill material would have to be imported and should conform to Ontario Provincial Standard Specifications (OPSS) for Granular A and Granular B Type II, depending on their use at the site.

Closure

The above and other related considerations are discussed in greater detail in the main body of the attached geotechnical report.

This executive summary is a brief synopsis of the geotechnical report and should not be read in lieu of reading the geotechnical report in its entirety.

1.0 Introduction

EXP Services Inc. (EXP) is pleased to present the results of the geotechnical investigation completed for the proposed residential development to be located at 85 Gemini Way (Lot B), Ottawa, Ontario (Figure 1). Authorization to proceed with this geotechnical investigation was provided by Centurion Appelt (1 Centrepoint) LP.

The proposed development will comprise of a six (6) storey wood framed building with two (2) level underground parking garage. The Architectural Drawing No. A4-2 titled, Building Section, dated September 15,2025 and prepared by Brouwer Architecture Inc. indicates that the proposed building may be supported by a raft (mat) foundation with design elevation of the lowest parking garage floor slab (the second level of the parking garage) at Elevation 79.70 m and top of raft foundation at Elevation 79.275 m. It is our understanding that consideration is also given to supporting the proposed building by footings.

A portion of the east side of the proposed new building will be located in close proximity to the west side of the neighboring existing building at 2140 Baseline Road. From the set of civil drawings, the site grading plan, Drawing No. C301, dated October 22,2025 (Revision No. 2) and prepared by LRL Engineering indicates there will be no grade raise at the site of the proposed building. The final grades will match the existing grades with some cut areas where the final grade will be lower than the existing grade.

The geotechnical investigation was undertaken to:

(a) Establish subsurface soil and groundwater conditions at six (6) boreholes located on the site,

(b) Provide site designation and classification for seismic design in accordance with the 2024 Ontario Building Code (OBC) and assess the potential for liquefaction of the subsurface soils during a seismic event,

(c) Comment on grade raise restrictions,

(d) Make recommendations regarding the most suitable type of foundations, founding depth and bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance at ultimate limit state (ULS) of the founding strata for the proposed new building and comment on the anticipated total and differential settlements of the recommended foundation type,

(e) Comment on slab-on-grade construction and requirements for perimeter and underfloor drainage systems,

(f) Provide soil parameters (for static and seismic conditions) for lateral earth pressure against subsurface (basement) walls,

(g) Comment on excavation conditions and dewatering requirements during construction,

(h) Provide pipe bedding requirements,

(i) Discuss backfilling requirements and the suitability of the on-site soil for backfilling purposes,

(j) Comment on the corrosion potential of the subsurface soils to buried concrete and steel; and

(k) Discuss tree planting restrictions.

The comments and recommendations given in this report are based on the assumption that the above described design concept will proceed into construction. If changes are made either in the design phase or during construction, this office must be retained to review these modifications. The result of this review may be a modification of our recommendations or it may require additional field or laboratory work to check whether the changes are acceptable from a geotechnical viewpoint.

2.0 Site Description

At the time of this geotechnical investigation, the site of the proposed residential development was used as an outdoor parking lot with paved and unpaved areas. The site is bounded by Baseline Road to the north, Gemini Way to the south, a high-rise building to the east and outdoor paved parking lot and office-type building to the west.

The topography of the site is flat as indicated by ground surface elevations at the borehole locations ranging from Elevation 86.59 m to Elevation 86.32 m.

3.0 Procedure

3.1 Borehole Fieldwork

The borehole fieldwork was undertaken in two (2) stages. The first stage was conducted on December 12 and 19, 2024 and consists of three (3) boreholes (Borehole Nos. 24-01 to 24-03) extending to auger refusal and termination depths of 12.3 m to 15.8 m below existing grade. The second stage was undertaken on October 15 and 16, 2025 and consists of three (3) boreholes (Borehole Nos. 25-01 to 25-03) extending to cone refusal and termination depths of 9.8 m and 12.6 m depths below existing grade. The borehole fieldwork was supervised on a full-time basis by a representative from EXP.

The locations and the geodetic elevations of the boreholes were established on site by EXP and are shown on the Borehole Location Plan, Figure 2.

The boreholes were cleared of private and public underground services, prior to the start of borehole drilling operations.

The boreholes were drilled using a CME-75 truck-mounted drill rig equipped with continuous flight hollow stem augers and soil sampling and bedrock coring capabilities. Auger samples were retrieved from the ground surface to a 0.6 m depth in Borehole Nos. 24-01 to 24-03. Borehole Nos. 25-01 to 25-03 were advanced by power augering technique (no sampling) from the ground surface to a 1.5 m depth. Standard penetration tests (SPTs) were performed in all the boreholes at 0.75 m to 1.5 m depth intervals with soil samples retrieved by the split-barrel sampler. A dynamic cone penetration tests (DCPT); unsampled) was conducted below the sampling depth to cone refusal depth in Borehole No. 25-02. The undrained shear strength of the clayey cohesive soils was measured by conducting in-situ vane tests at selected depth intervals in the boreholes and penetrometer tests on some of the recovered soil samples. The bedrock was cored in Borehole No. 24-03 by conventional rock coring method. A careful record of any sudden drops in the core barrel, colour of wash water and percentage of wash water return were recorded during the bedrock coring operation.

All the soil samples were examined in the field for textural classification, logged and preserved in labelled and identified plastic bags. Similarly, the bedrock cores were placed in labelled and identified bedrock core boxes and visually examined and logged in the field.

Nineteen (19) mm diameter standpipe and fifty (50) mm diameter monitoring wells with screened sections were installed in selected boreholes for long-term monitoring of the groundwater levels. The standpipe and monitoring wells were installed in accordance with EXP standard practice, and the installation configuration is documented on the respective borehole log. The boreholes were backfilled upon completion of drilling and the installation of the standpipe and monitoring wells.

On completion of the borehole fieldwork, the soil samples were transported to the EXP laboratory in Ottawa for laboratory testing of selected soil samples. The tested soil samples were classified by their main constituents in accordance with the Unified Soil Classification System (USCS) using the soil group name and symbol and by the modified Burmister Soil to classify the minor constituents of the soil using modifiers and adjectives such as trace and some.

3.2 Laboratory Testing Program

The geotechnical laboratory testing program is summarized in Table I.

Table I: Summary of Laboratory Testing Program

Type of Test	Number of Tests Completed
Soil Samples	
Moisture Content Determination	62
Grain Size Analysis	4
Atterberg Limit Determination	3
Corrosion Analysis (pH, sulphate, chloride and resistivity)	2
Rock Core Sections	
Unconfined Compressive Strength	2
Unit Weight Determination	2

3.3 Seismic Shear Wave Velocity Sounding Survey

A seismic shear wave velocity sounding survey was conducted at the site on September 25,2025 by Geophysics GPR International Inc. (GPR). The GPR seismic shear wave velocity sounding survey report is shown in Appendix A. As indicated in the GPR report, the survey was undertaken using the multi-channel analysis of surface waves (MASW), spatial auto correlation (SPAC) and seismic refraction methods.

4.0 Subsurface Conditions and Groundwater Levels

A detailed description of the subsurface conditions and groundwater levels from the boreholes are given on the attached Borehole Logs, Figure Nos. 3 to 8. The borehole logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions at the locations where sampling was conducted. The passage of time also may result in changes in the conditions interpreted to exist at the locations where sampling was conducted.

Boreholes were drilled to provide representation of subsurface conditions as part of a geotechnical exploration program and are not intended to provide evidence of potential environmental conditions.

It should be noted that the soil and bedrock boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling operations. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The Note on Sample Descriptions” preceding the borehole logs form an integral part of this report and should be read in conjunction with this report.

A review of the borehole logs indicates the following subsurface conditions with depth and groundwater level measurements.

4.1 Pavement Structure

Borehole Nos. 24-01 and 24-02 are located in a paved area where the pavement structure consists of 25 mm and 38 mm thick asphaltic concrete underlain by 700 mm and 725 mm thick granular fill base. The granular fill base consists of silty sand and gravel.

4.2 Topsoil

A surficial 50 mm thick topsoil layer was contacted in Borehole No. 24-03.

4.3 Fill

The pavement structure and topsoil in Borehole Nos. 24-01 to 24-03 are underlain by fill. Fill was inferred within the power-augered (no sampling) depths from ground surface to a 1.5 m depth in Borehole Nos. 25-01 to 25-03 and confirmed to exist in Borehole Nos. 25-01 and 25-03 below the 1.5 m inferred depth to 1.9 m and 2.2 m depths (Elevation 84.7 m and Elevation 84.3 m). In Borehole No. 25-02, the fill is inferred to extend to a 1.5 m depth (Elevation 84.8 m). The fill consists of silty sand and gravel with clay. The fill also contains topsoil inclusions, roots and rootlets. Based on the standard penetration test (SPT) N-values of 7 to 20, the fill is in a loose to compact state. The moisture content of the fill ranges from 15 percent to 56 percent

4.4 Silty Clay

A deep sensitive marine silty clay was contacted below the fill in all the boreholes. The silty clay was contacted at 1.2 m to 2.2 m depths (Elevation 85.2 m to Elevation 84.3 m) and extends to depths ranging from 9.8 m to 10.6 m (Elevation 76.5 m to Elevation 75.8 m) in Borehole Nos. 24-01 to 24-03. Borehole Nos. 25-01 to 25-03 terminated within the silty clay at a 9.8 m depth (Elevation 76.8 m to Elevation 76.5 m).

The sensitive marine silty clay consists of an upper desiccated weathered brown silty clay crust that exhibits good strength properties underlain by an un-desiccated unweathered grey silty clay of lower strength properties.

The upper and lower portions of the silty clay are discussed in the following sections of this geotechnical report.

4.4.1 Upper Brown Silty Clay Crust

The upper brown silty clay was encountered at 1.2 m to 2.2 m depths (Elevation 85.2 m to Elevation 84.3 m) and extends to depths ranging from 3.4 m to 4.6 m (Elevation 82.9 m to Elevation 81.8 m). The upper brown silty clay contains silty sand layers. The undrained shear strength of the brown silty clay crust ranges from 86 kPa to 200 kPa indicating the brown silty clay crust has a stiff to hard consistency. Locally, in Borehole No. 24-02 at an approximate 3.4 m depth (Elevation 82.9 m), the undrained shear strength of the brown silty clay is 48 kPa indicating a zone where the consistency of the silty clay is firm. The sensitivity of the brown silty clay ranges from 6.0 to 11.0 indicating the brown silty clay has a low to medium sensitivity based on the 2023 Fifth Edition of the Canadian Foundation Engineering Manual (CFEM). The natural moisture content of the brown silty clay crust ranges from 33 percent to 55 percent.

The results from the grain-size analysis and Atterberg limits determination conducted on one (1) sample of the brown silty clay are summarized in Table II. The grain-size distribution curve is shown in Figure 9.

Table II: Summary of Grain-Size Analysis and Atterberg Limit Determination
Upper Brown Silty Clay

Borehole No. (BH): Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)				Atterberg Limits (%)				Soil Classification
		Gravel	Sand	Silt	Clay	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	
BH 24-01: SS4	2.3-2.9	0	5	34	61	43	62	19	43	Silty Clay of High Plasticity (CH) – Trace Sand

A review of the test results indicates the soil may be classified as a silty clay of high plasticity (CH) with trace sand.

4.4.2 Lower Grey Silty Clay

The grey silty clay was contacted below the brown silty clay crust in all boreholes and extends to depths ranging from 9.8 m to 10.6 m (Elevation 76.5 m to Elevation 75.8 m) in Borehole Nos. 24-01 to 24-03. Borehole Nos. 25-01 to 25-03 terminated within the grey silty clay at a 9.8 m depth (Elevation 76.8 m to Elevation 76.5 m). The grey silty clay also contains silty sand layers. Based on undrained shear strength measurements of 34 kPa to 86 kPa, the grey silty clay has a firm to stiff consistency. Locally, in Borehole No. 25-02, the grey silty clay has a soft zone at an approximate 3.7 m depth (Elevation 82.6 m) as indicated by an undrained shear strength measurement of 24 kPa. The sensitivity of the grey silty clay ranges from 3.5 to 13.0 indicating the grey silty clay has a low to medium sensitivity based on the 2023 Fifth Edition CFEM. The natural moisture content of the grey silty clay is 24 percent to 62 percent.

The results from the grain-size analysis and Atterberg limits determination conducted on two (2) samples of the grey silty clay are summarized in Table III. The grain-size distribution curves are shown in Figures 10 and 11.

Table III: Summary of Grain-Size Analysis and Atterberg Limit Determination
Lower Grey Silty Clay

Borehole No. (BH) : SampleNo. (SS)	Depth (m)	Grain-Size Analysis (%)				Atterberg Limits (%)				Soil Classification
		Gravel	Sand	Silt	Clay	Moisture Content	Liquid Limit	Plastic Limit	Plasticity Index	
BH 24-01: SS9	7.6-8.2	0	20	43	37	54	31	12	19	Silty Clay of Medium Plasticity (CI): Sandy
BH 24-03: SS6	4.6-5.2	0	8	55	37	55	35	11	24	Silty Clay of Medium Plasticity (CI): Trace Sand

Based on a review of the laboratory test results, the soil may be classified as a silty clay of medium plasticity (CI) that is sandy to containing trace of sand.

4.5 Glacial Till

The silty clay in Borehole Nos. 24-01 to 24-03 is underlain by glacial till contacted at 9.8 m to 10.6 m depths (Elevation 76.5 m to Elevation 75.8 m). The glacial till extends to a 12.5 m depth (Elevation 73.9 m) in Borehole No. 24-03. The glacial till contains varying percentages of gravel, sand, silt and clay. The glacial till also contains cobbles, boulders and rock fragments. Based on standard penetration test (SPT) N-values of 48 and 74, the glacial till is in a dense to very dense state. Locally in Borehole No. 24-03 at an 11.0 m depth, the SPT N-value of the glacial till is 7, indicating a loose zone within the glacial till. At some depths the SPT N-value is high for low sampler penetration such as 50 for 50 mm of sampler penetration. This may be a result of the sampler contacting a cobble, boulder or rock fragment within the glacial till. The glacial till has a natural moisture content ranging from 8 to 21 percent.

The results from the grain-size analysis conducted on one (1) sample of the glacial are summarized in Table IV. The grain-size distribution curve is shown in Figure 12.

Table IV : Summary of Results form Grain Size Analysis – Glacial Till

Borehole No. (BH) – Sample No. (SS)	Depth (m)	Grain-Size Analysis (%)				Soil Classification
		Gravel	Sand	Silt	Clay	
BH 24-01-SS11	10.7-11.3	13	37	40	10	Silty Sand (SM): Some Gravel and Clay

Based on a review of the results of the grain-size analysis, the glacial till may be classified as a silty sand (SM) with some gravel and clay. The glacial till contains cobbles, boulders and rock fragments.

4.6 Inferred and Actual Limestone Bedrock

Based on auger and cone refusal criteria, inferred bedrock was encountered at 12.3 m to 12.8 m depths (Elevation 74.2 m to Elevation 73.5 m). The presence of bedrock was confirmed in Borehole No. 24-03 at a 13.2 m depth (Elevation 73.2 m) by bedrock coring operations. A bouldery glacial till or weathered bedrock was contacted above the bedrock in Borehole No. 24-03 from 12.5 m to 13.2 m depths (Elevation 73.9 m to Elevation 73.2 m). Photographs of the bedrock cores are shown in Figure 13.

Based on a review of the published bedrock geology map titled, Generalized Bedrock Geology Ottawa-Hull, Ontario and Quebec dated 1976, Geological Survey of Canada (Map 1508A), the site is underlain by limestone bedrock (with shaley partings) of the Ottawa formation.

The results of the coring of the bedrock indicate a total core recovery (TCR) of 88 percent and 100 percent. The rock quality designation (RQD) is 78 percent and 79 percent indicating the bedrock is of a good quality.

The unit weight and unconfined compressive strength of the selected core sections of the bedrock are summarized in Table V.

Table V: Summary of Unit Weight and Unconfined Compressive Strength Test Results Bedrock Cores

Borehole (BH) No.: Run No.	Depth (Elevation), m	Unit Weight (kN/m^3)	Unconfined Compressive Strength (MPa)	Classification of Rock with respect to Strength
BH 24-03: Run 2	14.1-14.3 (72.3-72.1)	26.2	83.4	Strong (R4)
BH 24-03: Run 3	14.8-14.9 (71.6-71.5)	26.5	118.2	Very Strong (R5)

A review of the test results in Table V indicates the strength of the rock may be classified as strong (R4) to very strong (R5) in accordance with the 2023 Fifth Edition Canadian Foundation Engineering Manual (CFEM).

4.7 Groundwater Levels

A summary of the groundwater level measurements taken on October 30,2025 in the boreholes equipped with monitoring well or standpipe is shown in Table VI.

Table VI: Summary of Groundwater Level Measurements

Borehole No. (BH)	Ground Surface Elevation (m)	Elapsed Time in Days from Date of Installation	Depth Below Ground Surface (Elevation), m
BH 24-01	86.47	10 Months and 18 Days	Inaccessible
BH 24-02	86.32	10 Months and 18 Days	4.3 (82.0)
BH 25-02	86.32	14 days	7.3 (79.0)

Based on a review of the groundwater level measurements from October 30,2025, the groundwater level is at 4.3 m and 7.3 m depths (Elevation 82.0 m and Elevation 79.0 m).

The groundwater levels were determined in the boreholes at the time and under the condition stated in this geotechnical report. Note that fluctuations in the level of groundwater may occur due to a seasonal variation such as precipitation, snowmelt, rainfall activities, and other factors not evident at the time of measurement and therefore may be at a higher level during wet weather periods.

5.0 Site Classification and Designation for Seismic Design and Liquefaction Potential of Soils

5.1 Site Classification and Designation for Seismic Design

A seismic shear wave velocity sounding survey was conducted at the site on September 25,2025 by Geophysics GPR International Inc. (GPR). The survey was undertaken using the multi-channel analysis of surface waves (MASW), spatial auto correlation (SPAC) and seismic refraction methods. The GPR seismic shear wave velocity sounding survey report is shown in Appendix A.

The results of the survey indicate that the average seismic shear wave velocity (V_{s30}) over a 30 m depth is 388 m/s. Based on a comparison of V_{s30} equal to 388 m/s with Sentence (2) in Section 4.1.8.4 of the 2024 Ontario Building Code (OBC), the site designation for seismic design is X_{388} which also corresponds to a Site Class C.

5.2 Liquefaction Potential of Soils

The subsurface soils are not considered to be susceptible to liquefaction during a seismic event.

6.0 Grade Raise Restrictions

From the set of civil drawings, the site grading plan, Drawing No. C301, dated October 22,2025 (Revision No. 2) and prepared by LRL Engineering indicates there will be no site grade raise for the proposed development. The final grades will match the existing grades with some cut areas where the final grade will be lower than the existing grade.

There will be no site grade raise for the proposed development which is considered to be acceptable from a geotechnical perspective. Should the grading plan change from no site grade raise to a site grade raise, EXP should be contacted to review the acceptability of the new site grade raise and based on the approved new site grade raise provide updated bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance values for the foundations of the proposed building.

7.0 Foundation Considerations

The proposed development will comprise of a six (6) storey wood framed building with two (2) level underground parking garage. The Architectural Drawing No. A4-2 titled, Building Section, dated September 15,2025 and prepared by Brouwer Architecture Inc. indicates that the proposed building may be supported by a raft (mat) foundation with design elevation of the lowest parking garage floor slab (the second level of the parking garage) at Elevation 79.70 m and top of raft foundation at Elevation 79.275 m. It is our understanding that consideration is also given to supporting the proposed building by footings. A portion of the east side of the proposed new building will be located in close proximity to the west side of the existing building located east of and next door to the proposed new building at 2140 Baseline Road.

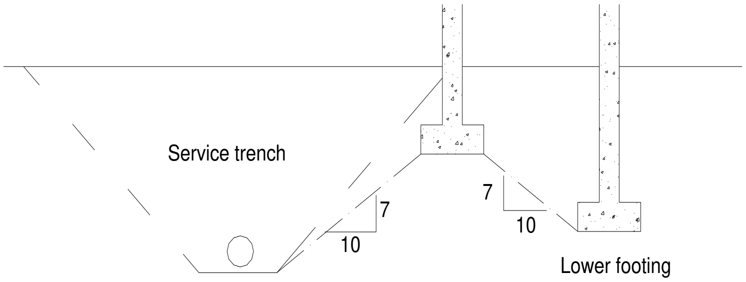
A portion of the east side of the proposed new building will be located in close proximity to the west side of the neighboring existing building at 2140 Baseline Road.

Based on a review of the borehole and design information for the proposed building, the proposed building may be supported by conventional strip and spread footings or by a raft (mat) foundation as discussed in the following sections of this geotechnical engineering report.

7.1 Footings

The proposed building may be supported by conventional strip and spread footings. For a top of footing designed at the same elevation as the top of the raft elevation noted above, Elevation 79.275 m, and assuming the footing thickness is 300 mm thick, the design elevation of the underside of the footings will be at Elevation 78.975 m. Based on the borehole information, at Elevation 78.975 m, the footings will be founded on the grey silty clay. Strip and spread footings founded on the grey silty clay at Elevation 78.975 m may be designed for a bearing pressure at serviceability limit state (SLS) of 120 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 180 kPa. The factored ULS value includes a geotechnical resistance value of 0.5. The SLS and factored ULS values are valid provided there is no grade raise on the site. The settlements of the footings designed for the above SLS value and properly constructed are expected to be within the normally tolerable limits of 25 mm total settlement and 19 mm differential settlement. Should the grading plan change from no site grade raise to a site grade raise or if the design elevation of the footings will be different from the elevation noted above, EXP should be contacted to review the acceptability of the new site grade raise and based on the approved new site grade raise provide updated bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance values for the footings of the proposed building.

Footings founded in soils at different elevations should be located such that the higher footings are set below a line drawn up at 10 horizontal to 7 vertical (10H:7V) from the near edge of the lower footing, as shown below. This concept should also be applied to service excavation, etc. to ensure that undermining is not a problem.



FOOTINGS NEAR SERVICE TRENCHES OR AT DIFFERENT ELEVATIONS

7.2 Raft (Mat) Foundation

The proposed building may be supported by a raft (mat) foundation designed as a drained structure with permanent perimeter and underfloor drainage systems. For a top of raft foundation designed at Elevation 79.275 m and assuming the raft foundation is 600 mm thick, the design elevation of the underside of the raft foundation will be at Elevation 78.675 m. Based on the borehole information, at Elevation 78.675 m, the raft foundation will be founded on the grey silty clay. A raft foundation founded on the grey silty clay at Elevation 78.675 m may be designed for a bearing pressure at serviceability limit state (SLS) of 145 kPa and factored geotechnical resistance at ultimate limit state (ULS) of 215 kPa. The factored ULS value includes a geotechnical resistance value of 0.5. The SLS and factored ULS values are valid provided there is no grade raise on the site. The settlements of the proposed raft foundation designed for the above SLS value and properly constructed are expected to be within the normally tolerable limits of 25 mm total settlement and 19 mm differential settlement. Should the grading plan change from no site grade raise to a site grade raise or if the design elevation of the raft foundation will be different from the elevation noted above, EXP should be contacted to review the acceptability of the new site grade raise and based on the approved new site grade raise provide updated bearing pressure at serviceability limit state (SLS) and factored geotechnical resistance values for the raft foundation of the proposed building.

7.2.1 Vertical Modulus of Subgrade Reaction

For the raft foundation founded at Elevation 78.675 m and designed for a bearing pressure at SLS, the vertical modulus of subgrade reaction may be taken as 6 MPa/m for design purposes.

7.3 Additional Comments for Footings and Raft Foundation

All footing beds and subgrade for the raft foundation should be examined by a geotechnical engineer to ensure that the founding soil is capable of supporting the bearing pressure at SLS and that the footing beds or subgrade for the raft foundation have been properly prepared.

It should be noted that the exposed silty clay subgrade surface is susceptible to disturbance due to movement of workers and construction traffic and the prevailing weather conditions during construction. To prevent disturbance to the soil subgrade, the approved footing beds or subgrade of the raft foundation should be covered or protected with a 50 mm thick concrete mud slab within the same day of approval.

The required earth cover to protect foundations of the proposed building will be satisfied for the footing or raft foundation founded at the above noted design elevations. However, for reference it is noted that a minimum of 1.5 m of earth cover should be provided to the exterior foundations on soil of heated structures to protect them from damage due to frost penetration. The frost cover should be increased to 2.1 m for unheated structures if snow will not be removed from their vicinity and to 2.4 m if snow will be removed from the vicinity of the structure. When earth cover is less than the minimum required, an equivalent thermal combination of earth cover and rigid insulation or rigid insulation alone should be provided. EXP can provide additional comments in this regard, if required.

7.4 Foundation Treatment Along East Wall Adjacent to 2140 Baseline Road

A portion of the east wall of the proposed new building at the site will be located in close proximity to the foundation wall and retaining wall (along the parking garage entrance ramp) of the existing building located next door at 2140 Baseline Road. The set of structural drawings (dated 10/06/2022 and prepared by Goodeve Structural Inc.) for the existing building at 2140 Baseline Road indicates the building has a four (4) storey underground parking garage and the building is supported by footings founded on bedrock. A portion of the retaining wall along the parking garage entrance ramp is also supported by footings founded on bedrock. The geotechnical engineering report by Paterson Group Inc. for 2140 Baseline Road dated November 28,2019 (PG4184-1 Revision 2) indicates the bedrock is at 10.0 m to 15.0 m depths.

Since a portion of the walls along the east side of the proposed new building will be located in close proximity to the foundation wall of the adjacent existing building and retaining wall (along the parking garage entrance ramp) of the existing building and the foundations will be set at a higher elevation than the footings of the existing building at 2140 Baseline Road, the higher foundations of the new building will exert a load onto the foundation walls of the existing building and the portion of the retaining wall along the parking garage entrance ramp. To prevent imposing additional load on the foundation and retaining wall of the existing building, it is recommended that the portion of the east wall of the proposed new building be supported by micropiles socketed into the underlying bedrock. The bedrock was contacted in Borehole No. 24-03 at a 13.2 m depth (Elevation 73.2 m). It is noted that the depth (elevation) to sound bedrock may vary at locations away from the borehole.

The limit of micropile along the east wall of the proposed new building is shown in Figure 2A.

The micropiles should be cased in the overburden soil, the zone of cobbles and boulders (‘bouldery’ glacial till) and into the upper level of the bedrock with the construction of the remainder of the micropile completed by drilling an uncased hole into the sound bedrock. Such a pile will carry the load in bond between the grout and the sound bedrock. The bond between the casing and the soil should be neglected. The casing of the micropile should extend into the upper level of the bedrock. The load carrying capacity of the micropile may be computed from the following expression:

$$P_{ult} = \pi \, \alpha_1 \, l_1 \, d_1$$

Where P_{ult} = Ultimate load carrying capacity of pile, kN

α_1 = The unfactored bond between the sound bedrock and grout at ultimate limit state (ULS) is 2000 kPa

l_1 = Length of the uncased portion of the pile socketed into the sound bedrock, m

d_1 = Diameter of drilled hole in bedrock, m

The computed ultimate capacity of the piles should be multiplied by a geotechnical resistance factor of 0.4 when computing the factored axial capacity in compression at ultimate limit state (ULS) and a geotechnical resistance factor of 0.3 when computing the factored axial capacity in tension at ultimate limit state (ULS).

Since a portion of the proposed building will be supported by a combination of either footings or a raft foundation and a portion of the east wall of the building will be supported by micropiles, control joints may be required to prevent differential settlement. The need for a joint, type and location of the joint required and deign details regarding the joint should be provided by the structural engineer.

It is noted that the pile borings should be cased in the overburden soil and the cobbles and boulder zone (‘bouldery’ glacial till) to prevent cave-in of these materials and to reduce the groundwater seepage into the pile holes. It is imperative that the holes for installation of the piles are cleaned properly so that the grout is in contact with the clean bedrock that is free of any soil smearing. All water should be pumped out from the pile borings prior to the placement of the grout.

It is noted that the overburden soil contains numerous boulders and cobbles (‘bouldery’ glacial till) which the installation contractor should take into consideration when selecting the method of drilling of the micro-piles. The contractor should anticipate the possibility of significant grout takes within the shaly zone of the bedrock during grouting operations. Also, water inflow into the drilled micro-piles holes should be expected.

It is recommended that the pile capacity should be confirmed by conducting pre-production or design performance tests on selected piles and/or proof load test on all piles.

8.0 Floor Slab and Drainage Requirements

The lowest floor of the parking garage may be designed and constructed as a slab-on-grade placed on a 200 mm thick well-packed 19 mm sized clear stone bed placed on a minimum 300 mm thick engineered fill pad set on the approved silty clay. The engineered fill pad should consist of Ontario Provincial Standard Specification (OPSS) Granular B Type II material compacted to 98 percent standard Proctor maximum dry density (SPMDD). The clear stone will minimize the capillary rise of moisture from the sub-soil to the floor slab. Alternatively, the clear stone layer may be replaced with a 200 mm thick bed of OPSS Granular A compacted to 100 percent SPMDD overlain by a vapour barrier. Adequate saw cuts should be provided in the floor slabs to control cracking.

The proposed building will require permanent perimeter and underfloor drainage systems.

The perimeter drainage system may consist of 100 mm diameter perforated pipe set on the footings and surrounded with 150 mm thick 19 mm sized clear stone that is fully wrapped or covered with an approved porous geotextile membrane, such as Terrafix 270R or equivalent. The underfloor drainage system may consist of 100 mm diameter perforated pipe or equivalent placed in parallel rows at 5 m to 6 m centres and at least 300 mm below the underside of the floor slab. The drains should be set on a 100 mm thick bed of 19 mm sized clear stone and covered on top and sides with 100 mm thick clear stone that is fully wrapped or covered with an approved porous geotextile membrane, such as Terrafix 270R or equivalent.

The perimeter and underfloor drainage systems should be connected to separate sumps equipped with backup (redundant) pumps and generators in case of mechanical failure and/or power outage, so that at least one system would be operational should the other fail.

The final exterior grade surrounding the proposed building should be sloped away from the building to prevent ponding of surface water close to the exterior walls of the building.

9.0 Lateral Earth Pressures Against Subsurface Walls

The subsurface basement walls of the proposed building should be backfilled with free draining material, such as OPSS Granular B Type II compacted to 95 percent SPMDD and equipped with a perimeter drainage system to prevent the buildup of hydrostatic pressure behind the walls. The walls will be subjected to lateral static and dynamic (seismic) earth forces. The expressions below assume free draining backfill material, a perimeter drainage system, level backfill surface behind the wall and vertical face on the back side of the wall.

For design purposes, the lateral static earth thrust against the subsurface walls may be computed from the following equation:

$$P = K_0 h (½ \gamma h + q)$$

where P = lateral earth thrust acting on the subsurface wall, kN/m

K_0 = lateral earth pressure at rest coefficient, assumed to be 0.5 for OPSS Granular B Type II backfill material

γ = unit weight of free draining granular backfill; Granular B Type II = 22 kN/m^3

h = depth of point of interest below top of backfill, m

q = surcharge load stress, kPa

The lateral dynamic thrust may be computed from the equation given below:

$$\Delta_{Pe} = \gamma H^2 \frac{a_h}{g} F_b$$

where Δ_{Pe} = dynamic thrust in kN/m of wall

H = height of wall, m

γ = unit weight of backfill material = 22 kN/m^3

$\frac{a_h}{g}$ = earth pressure coefficient = 0.336g for 2 percent probability of exceedance in 50 years from the 2020 National Building Code of Canada (NBCC) Seismic Hazard Tool for the site designation value of X_{388}

F_b = thrust factor = 1.0

The dynamic thrust does not take into account the surcharge load. The resultant force acts approximately at 0.63H above the base of the wall.

All subsurface walls should be properly waterproofed.

10.0 Excavations

10.1 Excess Soil Management

Ontario Regulation 406/19 specifies protocols that are required for the management and disposal of excess soils. As set forth in the regulation, specific analytical testing protocols need to be implemented and followed based on the volume of soil to be managed and the requirements of the receiving site. The testing protocols are specific as to whether the soils are stockpiled or in situ. In either scenario, the testing protocols are far more onerous than have been historically carried out as part of standard industry practices. These decisions should be factored in and accounted for prior to the initiation of the project-defined scope of work. EXP would be pleased to assist with the implementation of a soil management and testing program that would satisfy the requirements of Ontario Regulation 406/19.

10.2 Excavations

The excavations for the proposed building and installation of underground municipal services are anticipated to extend through the fill and into the silty clay to an 8.0 m depth below existing grade and are anticipated to be below the groundwater level.

The excavations may be undertaken by conventional heavy equipment.

All excavations must be undertaken in accordance with the Occupational Health and Safety Act (OHSA), Ontario Reg. 213/91. Based on the definitions provided in OHSA, the subsurface soils on site are considered to be Type 3 and as such must be cut back at 1H:1V from the bottom of the excavation. Within zones of seepage, the excavation side slopes are expected to slough and eventually stabilize at 2H:1V to 3H:1V from the bottom of the excavation.

If side slopes cannot be achieved due to space restrictions on site due to the proximity of open cut excavations to the property limits, existing infrastructure or to foundations of the adjacent existing building at 2140 Baseline Road, the new building construction would have to be undertaken within the confines of an engineered support system (shoring system). A conventional shoring system may consist of a soldier pile and timber lagging system or interlocking steel sheeting system.

The need for a shoring system, the most appropriate type of shoring system and the location, design and installation of the shoring system should be determined by the contractors bidding on this project. The design of the shoring system should be undertaken by a professional engineer experienced in shoring design and the installation of the shoring system should be undertaken by a contractor experienced in the installation of shoring systems. The shoring system should be designed and installed in accordance with latest edition of Ontario Regulation 213/91 under the OHSA and the 2006 Fourth Edition of the Canadian Foundation Engineering Manual (CFEM). The design of the shoring system will have to consider surcharge loads imposed on the shoring system by items such as adjacent existing footings, traffic and existing infrastructure. The shoring system will require lateral restraint provided by tiebacks consisting of rock anchors, rakers or an internal bracing system.

Base heave failure of the excavation for the proposed building is not anticipated for excavations that are properly dewatered and extend to an 8.0 m depth below existing grade into the firm to stiff silty clay.

The soil parameters that may be used in the design of the shoring system are summarized in Table VII.

Table VII – Soil Parameters for Shoring Design					
Soil Type	Total Unit Weight (kN/m^3)	Effective Unit Weight (kN/m3)	Angle of Internal Friction Angle (Degrees)	Active Lateral Earth Pressure Coefficient (K_A)	At-Rest Lateral Earth Pressure Coefficient (K_0)
FILL – Silty Sand and Gravel	22	12	30	0.33	0.50
Upper Brown Silty Clay	18	8	28	0.36	0.53
Lower Grey Silty Clay	16	6	28	0.36	0.53
Glacial Till	22	12	32	0.31	0.47

The shoring system as well as adjacent settlement sensitive structures and infrastructure should be monitored for movement (deflection) on a periodic basis during construction operations.

A pre-construction condition survey of adjacent neighboring structures and infrastructure within the influence zone of construction should be undertaken prior to the start of any construction activities and vibration monitoring of adjacent neighboring structures and infrastructure within the influence zone of construction should be undertaken during construction.

Many geologic materials deteriorate rapidly upon exposure to meteorological elements. Unless otherwise specifically indicated in this report, walls and floors of excavations must be protected from moisture, desiccation, and frost action throughout the course of construction.

A construction monitoring and control plan (CMCP) should be submitted by the contractor and discuss the impact and mitigation measures required to protect the existing backbone watermain located along Baseline Road north of the site for the proposed building. The backbone watermain is located north of the property line of the site of the proposed building and runs along and parallel with Baseline Road.

10.3 Guidelines for Construction Monitoring and Control Plan for Backbone Watermain

Geotechnical guidelines for the preparation of the construction monitoring control plan (CMCP) for the backbone watermain are as follows:

- Review drawings to determine the location and obvert/invert depths of the backbone watermain relative to the site of the proposed new building.
- Once the location and obvert/invert of the backbone watermain are known, a work plan should be prepared discussing the excavation for the new building in the vicinity of the backbone watermain and the proposed method to protect the backbone watermain during construction of the new building. The work plan should be submitted to the civil and geotechnical consultants and to the City of Ottawa for review. Excavation work for the proposed new building should not proceed until the work plan has been reviewed and approved by all parties. The work plan may consist of the following items:
    - Pre-construction condition survey of the watermain should be undertaken prior to the start of any construction activities.
    - Detailed study should be undertaken to identify and confirm the location and depths of existing underground services and the backbone watermain relative to the location of the excavation of the new building. 'Hydro-vac' method instead of excavating with heavy

construction equipment may be used to expose the backbone watermain and other existing underground services to confirm their location relative to the location of the excavation for the new building.

- Settlement monitoring program of the backbone watermain using surface and in-ground settlement monitoring survey points of the backbone watermain should be undertaken along the centreline of the watermain and at offsets to the watermain. The ground movements should be monitored at regular intervals during construction of the proposed building and the settlement monitoring program should include definitions for baseline readings, review and alert levels and plan of action and reporting process for review and alert levels.
- Vibration monitoring program at the existing backbone watermain should be undertaken during the construction of the new building to ensure that the vibrations generated during construction activities do not exceed the vibration limits applicable for the existing backbone watermain. For guidance, the City of Ottawa maximum peak particle velocity limits are provided in Table VIII.

Table VIII: Maximum Peak Particle Velocity Values
(City of Ottawa Special Provisions (SP) No. 1201)

Element	Frequency (Hz)	Peak Particle Velocity, PPV (mm/s)
Structures and Pipelines	Less than or equal to 40	20
	>40	50
Concrete and Grout less than 72 hours from placement	N/A	10

10.4 De-Watering Requirements

Seepage of the surface and subsurface water into unshored and shored excavations is anticipated. However, it should be possible to remove any water entering the excavations by collecting it in perimeter ditches or low points and pumping from sumps. In areas of high infiltration or in areas where more permeable soil layers may exist, such as the silty sand layers in the silty clay, a higher seepage rate should be anticipated and will require high-capacity pumps to keep the excavation dry.

For construction dewatering, an Environmental Activity and Sector Registry (EASR) approvals shall be obtained for water takings greater than 50 m3 per day. Since July 2025, any volume of pumping greater than 50m3/day will require to be registered as EASR. A hydrogeological assessment report, water taking and discharge plans are required for EASR registration.

Although this investigation has estimated the groundwater levels at the time of the fieldwork, and commented on dewatering and general construction problems, conditions may be present which are difficult to establish from standard boring and excavating techniques and which may affect the type and nature of dewatering procedures used by the contractor in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile, thin layers of soil with large or small permeabilities compared with the soil mass, etc. Only carefully controlled tests using pumped wells and observation wells will yield the quantitative data on groundwater volumes and pressures that are necessary to adequately engineer construction dewatering systems.

11.0 Impact on Adjacent Structures and Infrastructure

Based on the assumption that the surrounding existing building foundations and infrastructure are supported by the firm to stiff silty clay or limestone bedrock, the lowering of the groundwater level over the short-term excavation period and over the long-term for the proposed new building is not anticipated to negatively impact existing adjacent surrounding structures and infrastructure.

12.0 Pipe Bedding Requirements

The municipal services are anticipated to extend into the fill, silty clay and limestone bedrock.

The bedding for the underground services including material specifications, thickness of cover material and compaction requirements should conform to municipal requirements and/or Ontario Provincial Standard Specification and Drawings (OPSS and OPSD).

It is recommended that the pipe bedding be 300 mm thick and consist of OPSS Granular A. The bedding material should be placed along the sides and on top of the pipe to provide a minimum cover of 300 mm. The bedding should be compacted to at least 98 percent of the SPMDD.

The bedding thickness may be further increased in areas where the subgrade becomes disturbed or in areas of existing fill. Trench base stabilization techniques, such as the removal of loose/soft material, placement of additional sub-bedding, consisting of Ontario Provincial Standard Specification (OPSS) Granular B Type II compacted to 98 percent SPMDD and completely wrapped in a non-woven geotextile, may be used if trench base disturbance becomes a problem in wet or soft/loose areas.

To minimize settlement of the pavement structure over services trenches, the trench backfill material within the frost zone, to 1.8 m depth below final grade, should match the existing material along the trench walls to minimize differential frost heaving of the subgrade soil, provided this material is compactible. Otherwise, frost tapers may be required.

The municipal services should be installed in short open trench sections that are excavated and backfilled the same day.

13.0 Backfilling Requirements and Suitability of On-site Soils for Backfilling Purposes

The materials that will be excavated include asphaltic concrete, fill, silty clay, glacial till and limestone bedrock. These materials are not considered suitable for re-use as backfill material for the proposed new building and should be discarded.

Therefore, it is anticipated that the majority of the fill required for use as backfill material would have to be imported and should conform to Ontario Provincial Standard Specifications (OPSS) for Granular A and Granular B Type II, depending on their use at the site.

14.0 Subsurface Concrete and Steel Requirements

Chemical tests limited to pH, sulphate, chloride and resistivity were undertaken on two (2) soil samples. A summary of the test results is shown in Table IX. The laboratory certificate of analysis report by AGAT is shown in Appendix B.

Table IX: Corrosion Test Results - Soil

Borehole: Sample No.	Depth (m)	Soli Type	pH	Sulphate (%)	Chloride (%)	Resistivity (ohm-cm)
BH 24-02: SS7	4.6 – 5.2	Grey Silty Clay	7.63	0.0123	0.0532	826
BH 24-02: SS11	10.7 – 11.3	Glacial Till	8.14	0.0209	0.0838	709

The test results indicate the grey silty clay and glacial till have a negligible sulphate attack on subsurface concrete. The concrete should be designed in accordance with CSA A.23.1:24/CSA A23.2:24.

The results of the resistivity tests indicate that the grey silty clay and glacial till are corrosive to bare steel as per the National Association of Corrosion Engineers (NACE). Appropriate measures should be taken to protect the bare buried steel from corrosion.

.

15.0 Tree Planting Restrictions

The site of the proposed building is underlain by sensitive marine clay. The laboratory test results of the marine clay were compared with the document titled, "Tree Planting in Sensitive Marine Clay Soils – 2017 City of Ottawa Guidelines (2017 Guidelines)" and indicate the silty clay has a high potential for soil volume change. For soils that have a high potential for soil volume change, reference is made to the City of Ottawa 2005 Clay Soils Policy for tree planting restrictions and setbacks.

A landscape architect should be consulted to ensure the setbacks and tree planting restrictions are in accordance with the City of Ottawa 2005 Clay Soils Policy.

16.0 General Comments

The comments given in this geotechnical report are intended only for the guidance design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc., would be much greater than has been carried out for the design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

The information contained in this geotechnical report is not intended to reflect on environmental aspects of the soils. Should specific information be required, including for example, the presence of pollutants, contaminants or other hazards in the soil, additional testing may be required.

We trust that the information contained in this geotechnical report will be satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

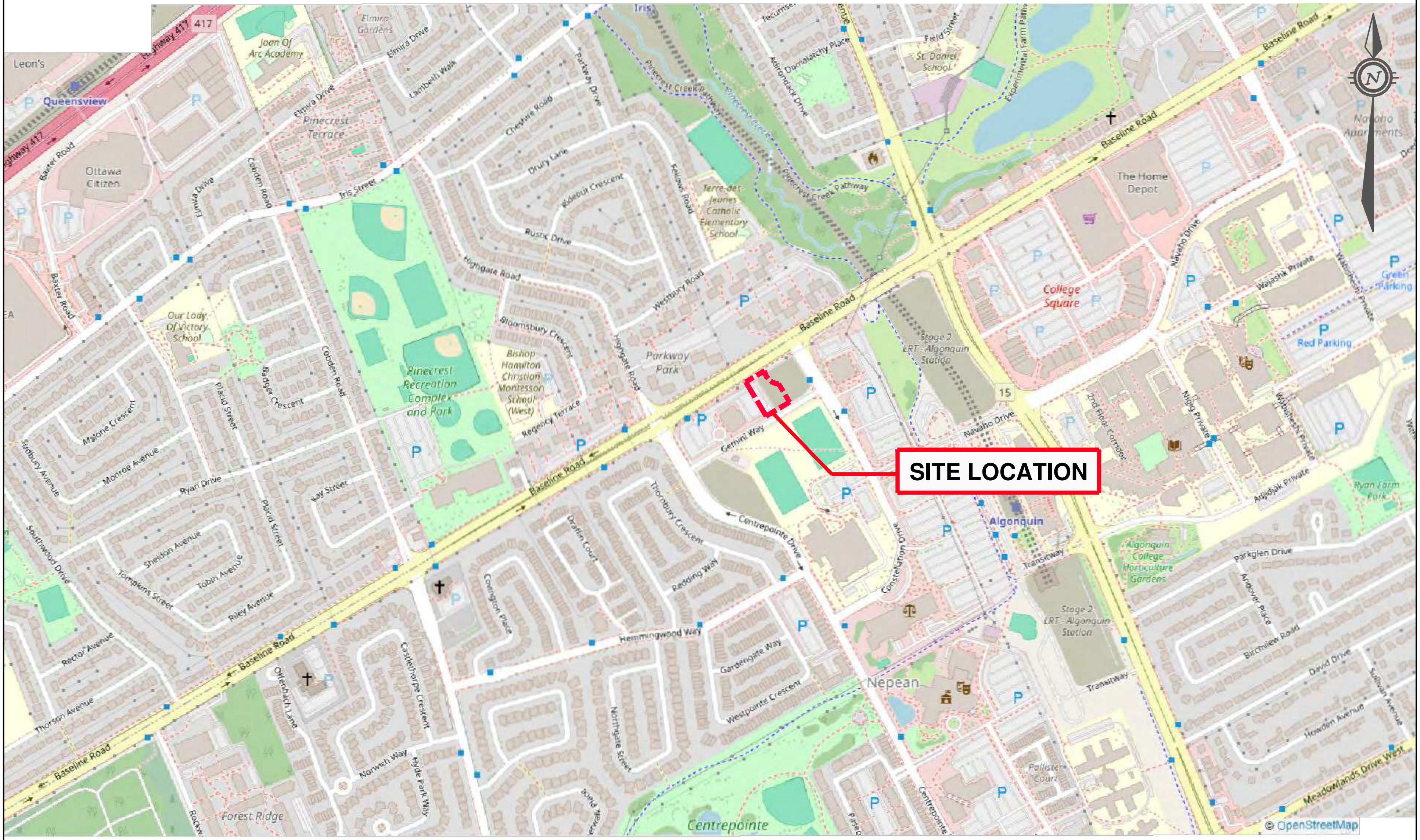
Sincerely,

Susan M. Potyondy, P.Eng.
Senior Geotechnical Engineer
Earth & Environment

LICENSED PROFESSIONAL ENGINEER
06.11.2025
S.M. POTYONDY
37232568
PROVINCE OF ONTARIO

Ismail M. Taki, M.Eng., P.Eng.
Senior Manager, Eastern Region
Earth & Environment

Figures



Source Map: *Open Street Map*, 2024

ORIGINAL SHEET SIZE = 11" X 8.5"

0 100m 200m 400m

HORIZONTAL 1:10,000

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

www.exp.com

DESIGN IT

DRAWN MS

DATE OCTOBER 2025

FILE NO OTT-24014796-A0

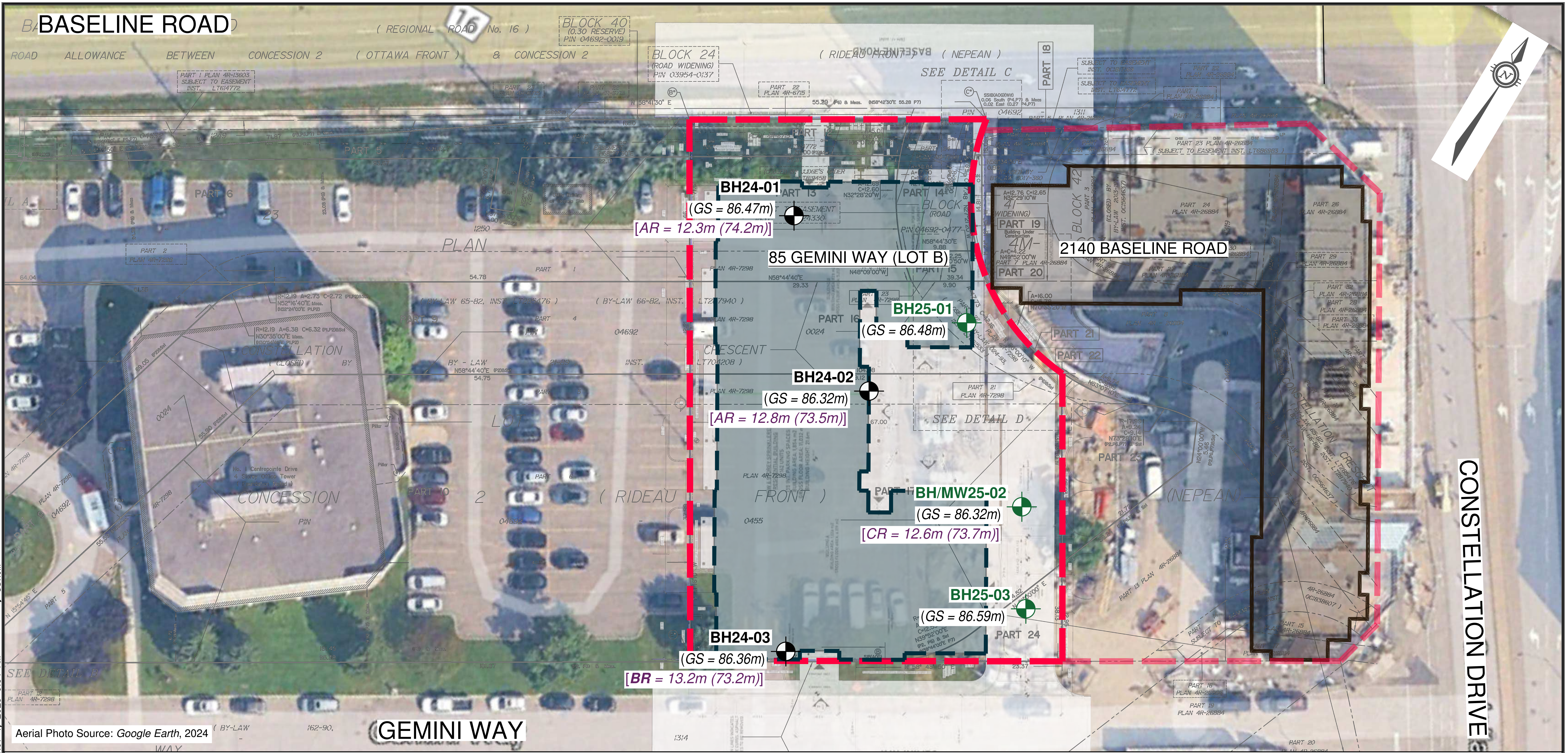
GEOTECHNICAL INVESTIGATION

PROPOSED RESIDENTIAL DEVELOPMENT
85 GEMINI (LOT B), OTTAWA, ONTARIO
SITE LOCATION PLAN

SCALE 1:10,000

SKETCH NO FIG 1

Filename: \\exp\data\OTT\OTT-24014796-A0\60 Execution\65 Drawings\OTT-24014796-A0_Geo_85-Gemilni-Way-Lot-B_Oct-2025_MS.dwg Plotted by: SavicM
Last Saved: Nov 4, 2025 9:53 AM Last Plotted: Nov 4, 2025 9:54 AM



LEGEND

▬ ▬	PROPERTY LINE
▨	PROPOSED NEW BUILDING APPROX. FOOTPRINT
▨	EXISTING BUILDING APPROX. FOOTPRINT
BH24-01 ⊕	BOREHOLE NO. & LOCATION (*EXP*, 2024)
BH 25-01 ⊕	BOREHOLE NO. & LOCATION (*EXP*, 2025)
(*GS = 86.47m*)	GROUND SURFACE ELEVATION (m)
[***BR** = 73.2m*]	BEDROCK DEPTH (ELEVATION) (m)
[*AR = 74.2m*]	AUGER REFUSAL DEPTH (ELEVATION) (m)
[*CR = 73.96m*]	CONE REFUSAL DEPTH (ELEVATION) (m)

NOTES:

1. THE BOUNDARIES, SOIL AND BEDROCK TYPES HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES THEY ARE ASSUMED AND MAY BE SUBJECT TO CONSIDERABLE ERROR.
2. SOIL SAMPLES AND BEDROCK CORES WILL BE RETAINED IN STORAGE FOR THREE MONTHS AND THEN DESTROYED UNLESS THE CLIENT ADVISES THAT AN EXTENDED TIME PERIOD IS REQUIRED.
3. BOREHOLE ELEVATIONS SHOULD NOT BE USED TO DESIGN BUILDING(S) OR FLOOR SLABS OR PARKING LOT(S) GRADES.
4. TOPSOIL AND ASPHALT QUANTITIES SHOULD NOT BE ESTABLISHED FROM THE INFORMATION AT THE BOREHOLE LOCATIONS.
5. THIS DRAWING FORMS PART OF THE REPORT PROJECT NUMBER AS REFERENCED AND SHOULD BE USED ONLY IN CONJUNCTION WITH THIS REPORT.
6. BASE PLAN OF SURVEY INFORMATION PRODUCED BY: *ANNIS O'SULLIVAN, VOLLEBEKK LTD.* , BLOCK 39 AND PART OF BLOCKS 22, 23 AND PART OF BLOCK 41, REGISTERED PLAN 4M-623 AND PART OF LOT 35 CONCESSION (RIDEAU FRONT), DATED: JULY 11, 2024.
7. BASE SITE PLAN PRODUCED BY: *BROUWER ARCHITECTURE* , PROJECT NO.: 2433, SHEET NO.: A1-1, DATED: 2025-09-17

ORIGINAL SHEET SIZE = 17" X 11"

0 5m 10m 20m

HORIZONTAL 1:500

exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6
www.exp.com

DESIGN	IT
DRAWN	MS
DATE	OCTOBER 2025
FILE NO	OTT-24014796-A0

GEOTECHNICAL INVESTIGATION

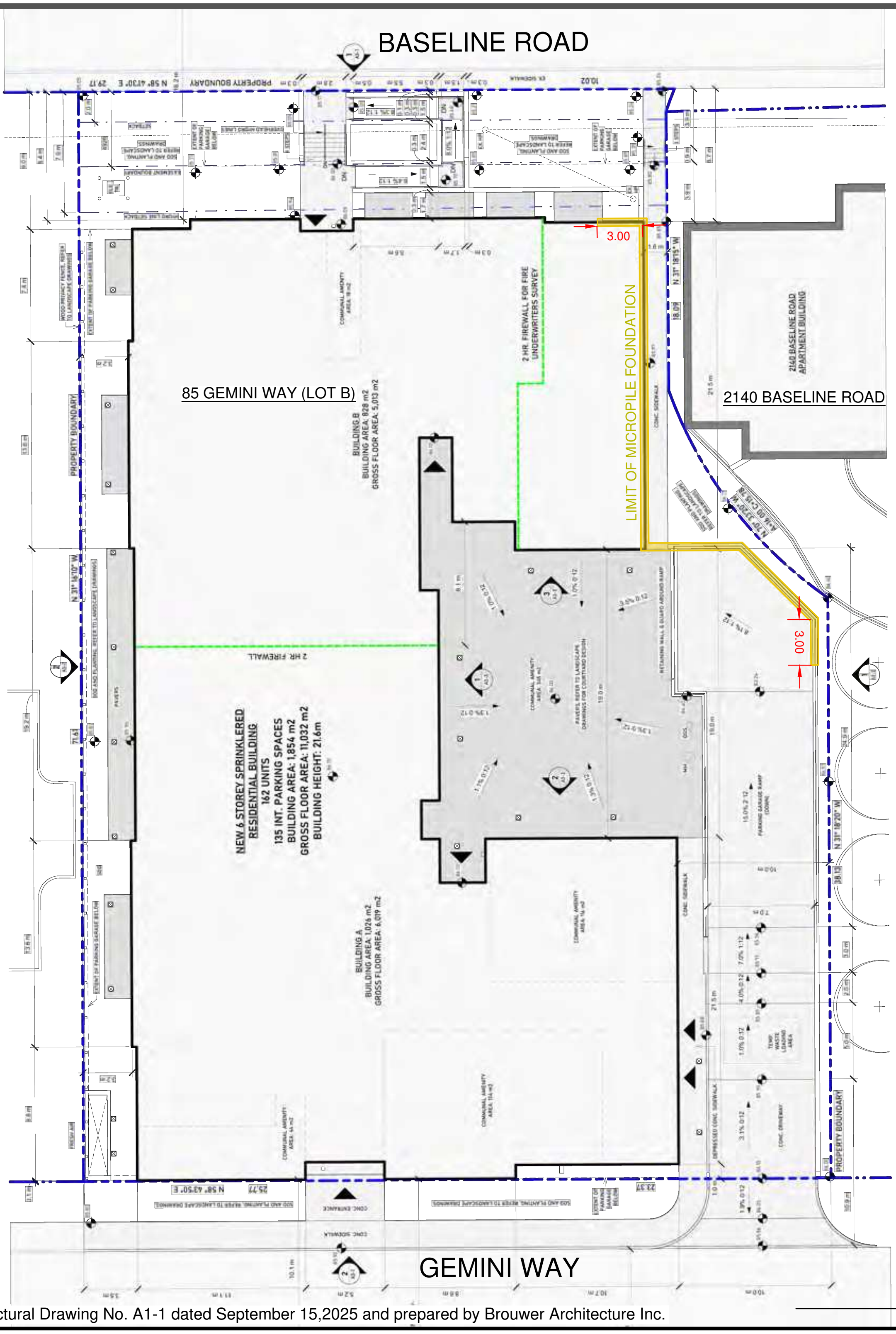
PROPOSED RESIDENTIAL DEVELOPMENT
85 GEMINI WAY (LOT B), OTTAWA, ONTARIO
BOREHOLE LOCATION PLAN

SCALE 1:500

SKETCH NO FIG 2

Filename: \\exp\data\OTT\OTT-24014796-A0\60 Execution\65 Drawings\OTT-24014796-A0_Geo_85-Gemilni-Way-Lot-B_Oct-2025_MS.dwg
Last Saved: Nov 4, 2025 9:53 AM Last Plotted: Nov 4, 2025 9:54 AM Plotted by: SavicM

Filename: \\exp\data\OTT\OTT-24014796-A0\60 Execution\65 Drawings\OTT-24014796-A0_Geo_85-Gemini-Way-Lot-B_Oct-2025_MS.dwg
Last Saved: Nov 4, 2025 9:59 AM Last Plotted: Nov 4, 2025 10:50 AM Plotted by: SavicM



Reference: Architectural Drawing No. A1-1 dated September 15,2025 and prepared by Brouwer Architecture Inc.

LEGEND

🟨 LIMIT OF MICROPILE FOUNDATION

0 5m 10m 30m

HORIZONTAL 1:750

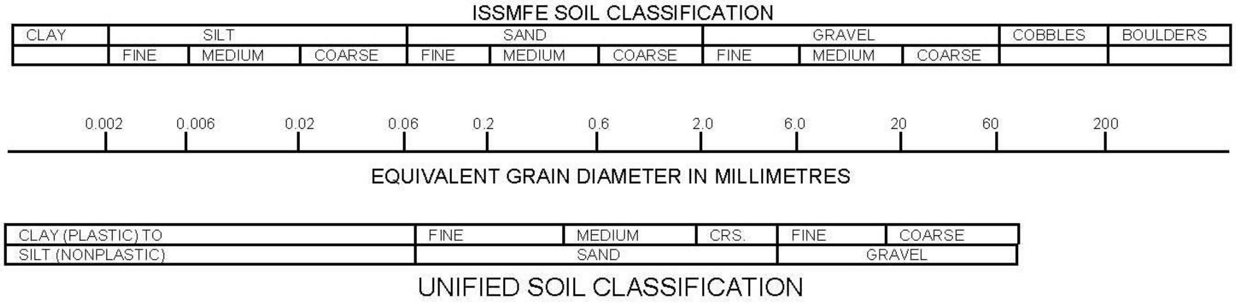
exp Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

www.exp.com

DESIGN IT	GEOTECHNICAL INVESTIGATION		SCALE 1:750
DRAWN MS			
DATE OCTOBER 2025	PROPOSED RESIDENTIAL DEVELOPMENT 85 GEMINI WAY (LOT B), OTTAWA, ONTARIO LIMIT OF MICROPILE FOUNDATION		SKETCH NO FIG 2A
FILE NO OTT-24014796-A0			

Notes On Sample Descriptions

1. All sample descriptions included in this report follow the Canadian Foundations Engineering Manual soil classification system. This system follows the standard proposed by the International Societv for Soil Mechanics and Foundation Engineering. Laboratory grain size analyses provided by **exp** Services Inc. also follow the same system. Different classification systems may be used by others; one such system is the Unified Soil Classification. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



2. Fill: Where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc., none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Log of Borehole BH24-01

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Location: 85 Gemini Way (Lot B), Ottawa, Ontario

Figure No. 3

Page. 1 of 1

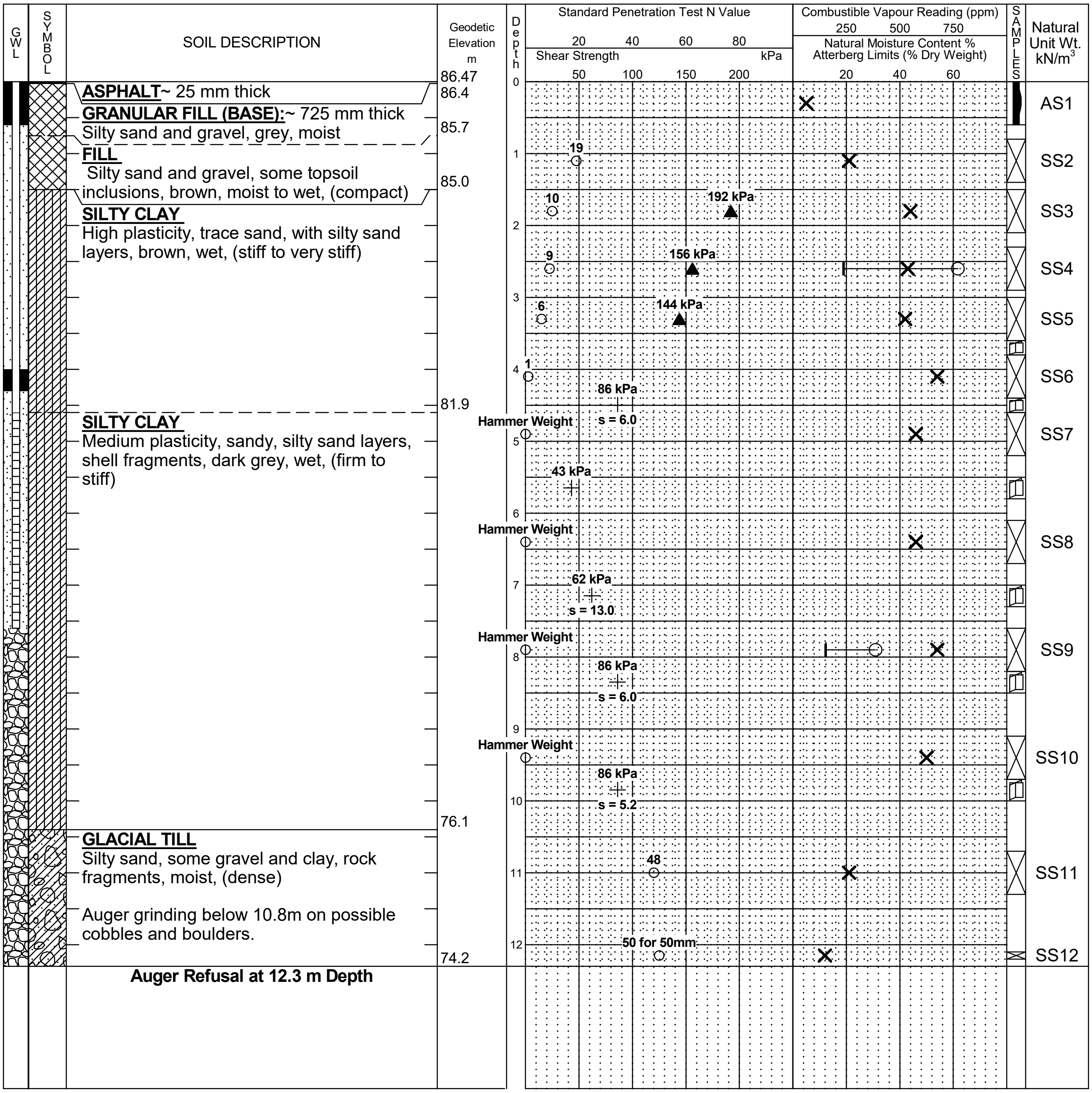
Date Drilled: 'December 12, 2024

Drill Type: CME 75 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: AN Checked by: IT

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



ASPHALT~ 25 mm thick

GRANULAR FILL (BASE):~ 725 mm thick
Silty sand and gravel, grey, moist

FILL
Silty sand and gravel, some topsoil inclusions, brown, moist to wet, (compact)

SILTY CLAY
High plasticity, trace sand, with silty sand layers, brown, wet, (stiff to very stiff)

SILTY CLAY
Medium plasticity, sandy, silty sand layers, shell fragments, dark grey, wet, (firm to stiff)

GLACIAL TILL
Silty sand, some gravel and clay, rock fragments, moist, (dense)

Auger grinding below 10.8m on possible cobbles and boulders.

Auger Refusal at 12.3 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	4.3	12.3
January 10 , 2025	Inaccessible	
'October 30, 2025	Inaccessible	

CORE DRILLING RECORD

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

Log of Borehole BH24-02

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Location: 85 Gemini Way (Lot B), Ottawa, Ontario

Figure No. 4

Page. 1 of 1

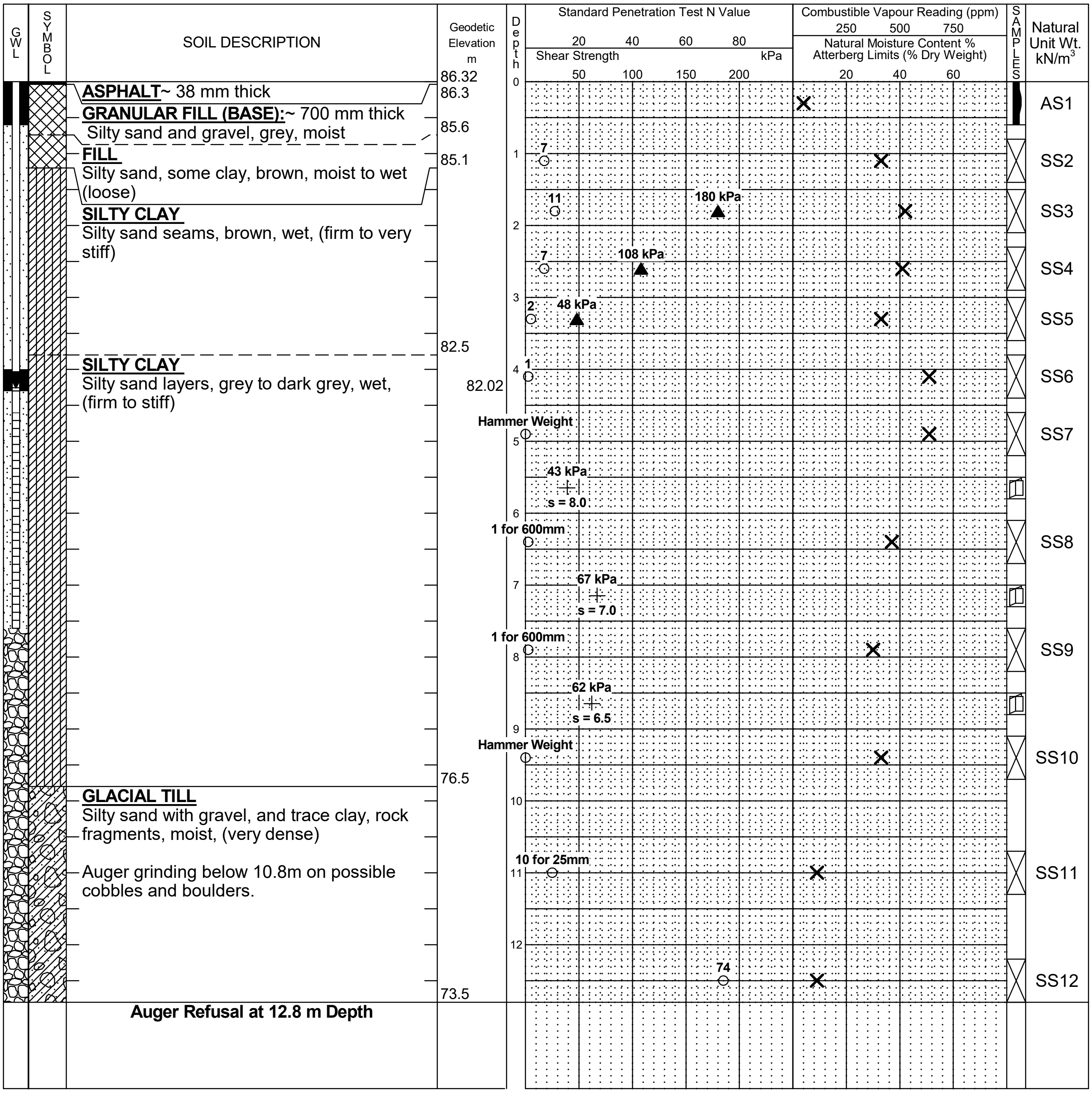
Date Drilled: 'December 12, 2024

Drill Type: CME 75 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: AN Checked by: IT

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



ASPHALT~ 38 mm thick

GRANULAR FILL (BASE):~ 700 mm thick
Silty sand and gravel, grey, moist

FILL
Silty sand, some clay, brown, moist to wet (loose)

SILTY CLAY
Silty sand seams, brown, wet, (firm to very stiff)

SILTY CLAY
Silty sand layers, grey to dark grey, wet, (firm to stiff)

GLACIAL TILL
Silty sand with gravel, and trace clay, rock fragments, moist, (very dense)

Auger grinding below 10.8m on possible cobbles and boulders.

Auger Refusal at 12.8 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 50mm diameter monitoring well installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
Completion	4.3	12.7
January 10 , 2024	7.5	
'October 30, 2025	4.3	

CORE DRILLING RECORD

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

Log of Borehole BH24-03

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Location: 85 Gemini Way (Lot B), Ottawa, Ontario

Figure No. 5

Page. 1 of 2

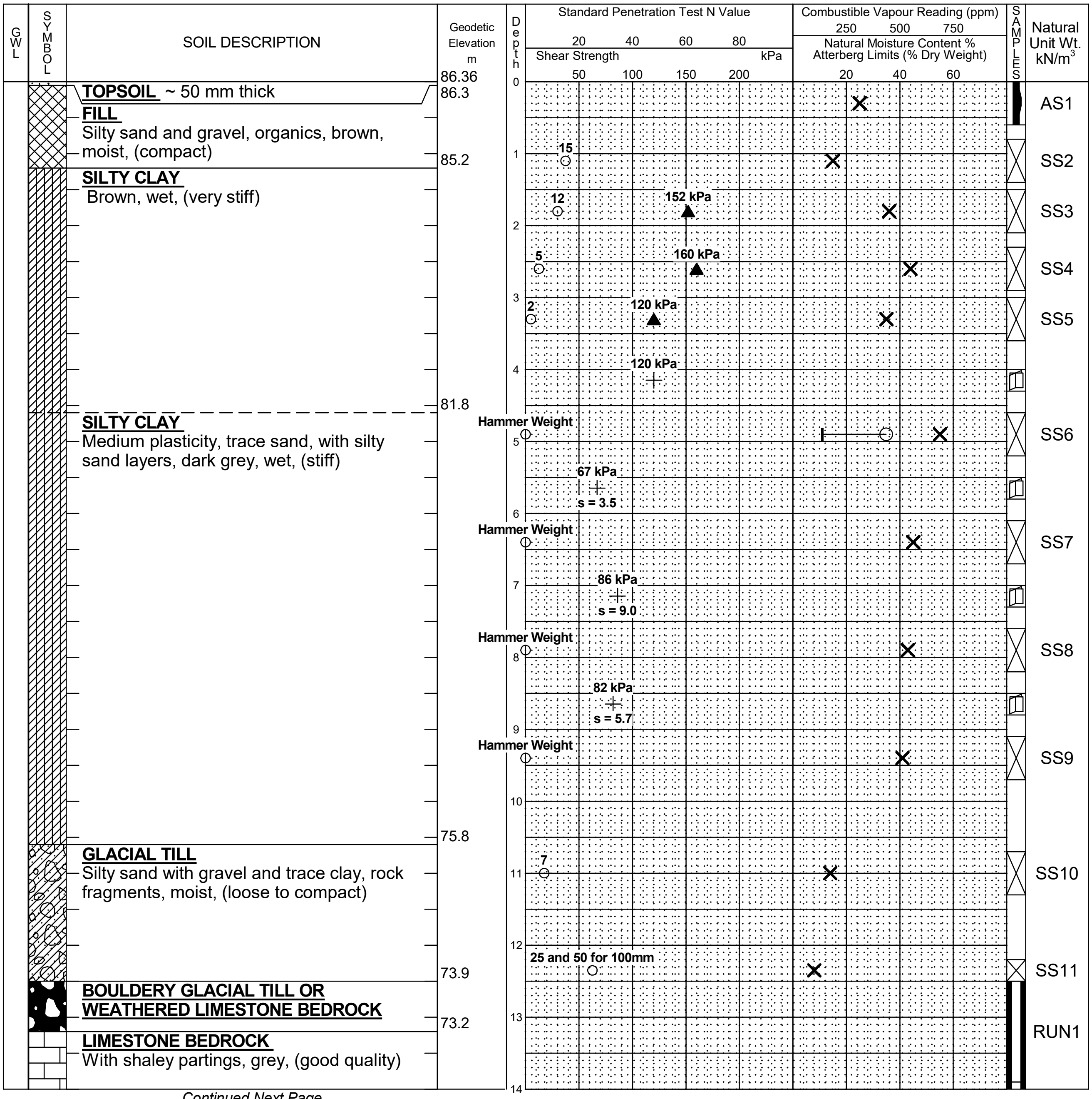
Date Drilled: 'December 19, 2024

Drill Type: CME 75 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: AN Checked by: IT

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



86.36

TOPSOIL ~ 50 mm thick — 86.3

FILL
Silty sand and gravel, organics, brown, moist, (compact) — 85.2

SILTY CLAY
Brown, wet, (very stiff) — 81.8

SILTY CLAY
Medium plasticity, trace sand, with silty sand layers, dark grey, wet, (stiff) — 75.8

GLACIAL TILL
Silty sand with gravel and trace clay, rock fragments, moist, (loose to compact) — 73.9

BOULDERY GLACIAL TILL OR WEATHERED LIMESTONE BEDROCK — 73.2

LIMESTONE BEDROCK
With shaley partings, grey, (good quality)

Continued Next Page

NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	12.5 - 13.9	68	0
2	13.9 - 14.7	88	79
3	14.7 - 15.8	100	78

Log of Borehole BH24-03

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Figure No. 5

Page. 2 of 2

GWL	SYMBOL	SOIL DESCRIPTION	Geodetic Elevation m	Depth	Standard Penetration Test N Value 20 40 60 80 Shear Strength kPa 50 100 150 200	Combustible Vapour Reading (ppm) 250 500 750 Natural Moisture Content % Atterberg Limits (% Dry Weight) 20 40 60	SAMPLES	Natural Unit Wt. kN/m^3
		LIMESTONE BEDROCK With shaley partings, grey, (good quality) *(continued)*	72.36 70.6	14 15				RUN2 RUN3
		Borehole Terminated at 15.8 m Depth						

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

Run No.	Depth (m)	% Rec.	RQD %
1	12.5 - 13.9	68	0
2	13.9 - 14.7	88	79
3	14.7 - 15.8	100	78

Log of Borehole BH25-01

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Location: 85 Gemini Way (Lot B), Ottawa, Ontario

Figure No. 6

Page. 1 of 1

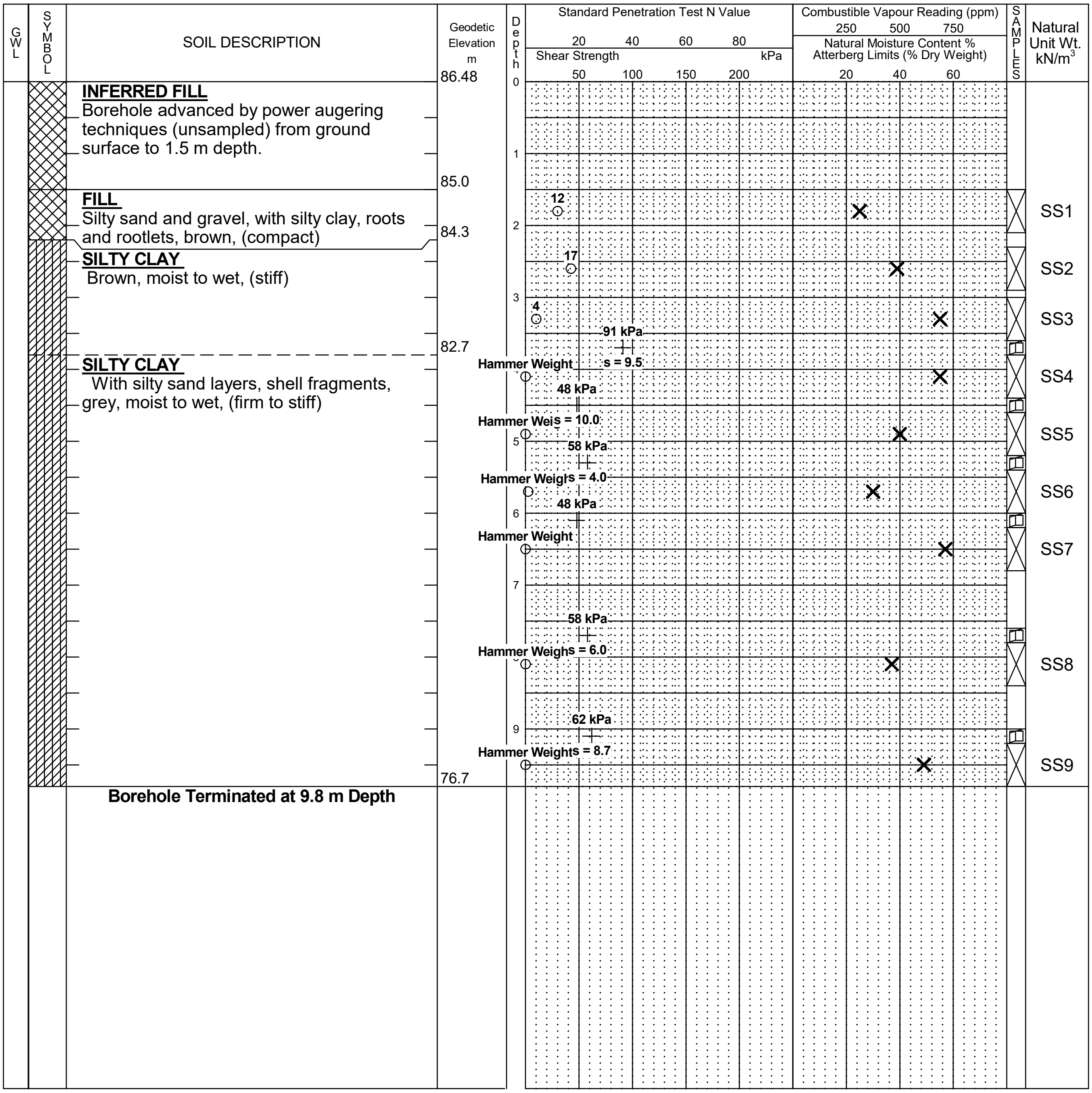
Date Drilled: 'October 15, 2025

Drill Type: CME 75 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: SA Checked by: SMP

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



G W L	S Y M B O L	SOIL DESCRIPTION	Geodetic Elevation m
			86.48
		INFERRED FILL Borehole advanced by power augering techniques (unsampled) from ground surface to 1.5 m depth.	85.0
		FILL Silty sand and gravel, with silty clay, roots and rootlets, brown, (compact)	84.3
		SILTY CLAY Brown, moist to wet, (stiff)	82.7
		SILTY CLAY With silty sand layers, shell fragments, grey, moist to wet, (firm to stiff)	76.7
		Borehole Terminated at 9.8 m Depth	

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

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

CORE DRILLING RECORD

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

Log of Borehole BH25-02

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Location: 85 Gemini Way (Lot B), Ottawa, Ontario

Figure No. 7

Page. 1 of 1

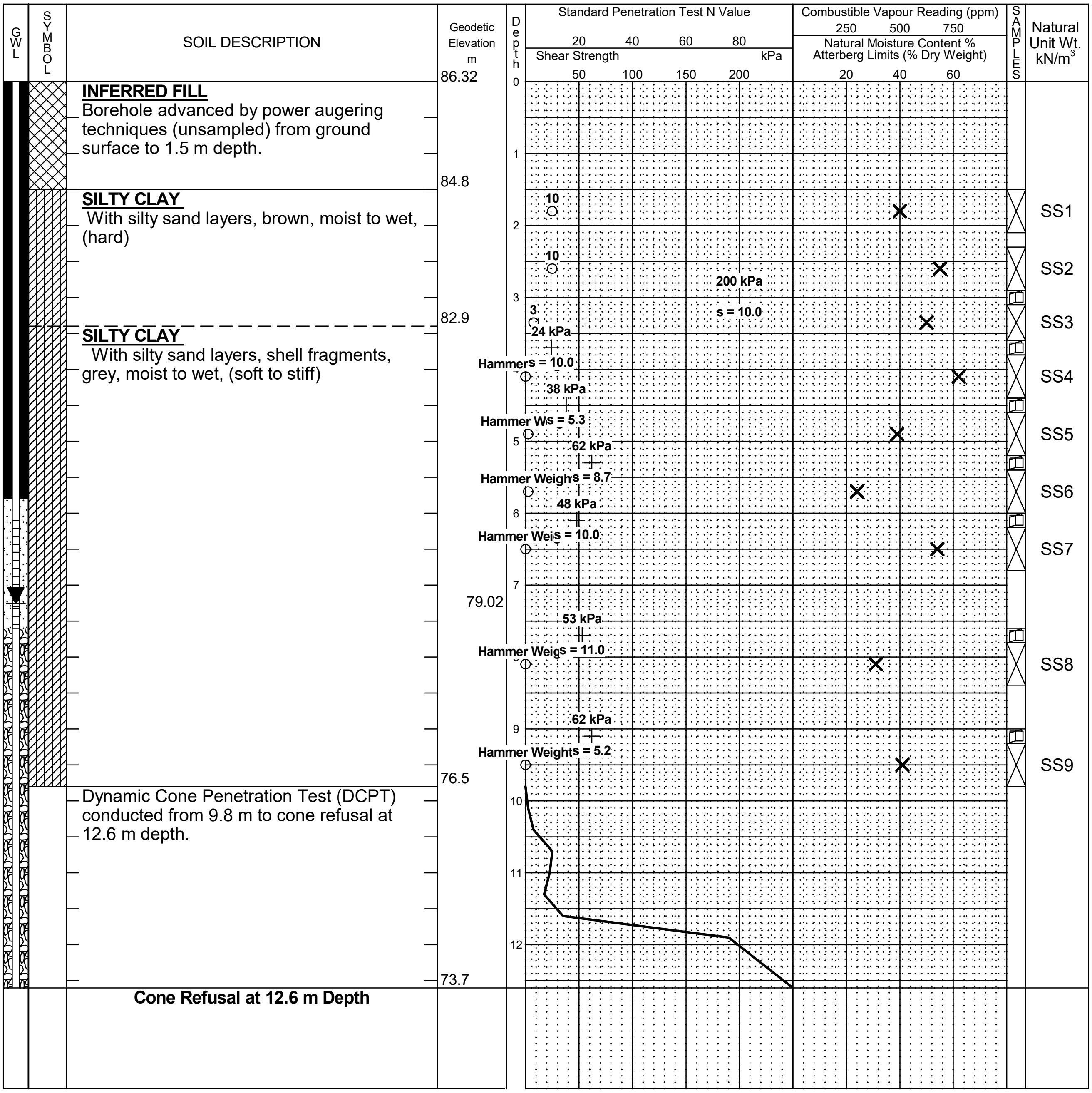
Date Drilled: 'October 16, 2025

Drill Type: CME 75 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: SA Checked by: SMP

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



INFERRED FILL
Borehole advanced by power augering techniques (unsampled) from ground surface to 1.5 m depth.

SILTY CLAY
With silty sand layers, brown, moist to wet, (hard)

SILTY CLAY
With silty sand layers, shell fragments, grey, moist to wet, (soft to stiff)

Dynamic Cone Penetration Test (DCPT) conducted from 9.8 m to cone refusal at 12.6 m depth.

Cone Refusal at 12.6 m Depth

NOTES:

1. Borehole data requires interpretation by EXP before use by others
2. A 19 mm diameter piezometer installed as shown.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

Date	Water Level (m)	Hole Open To (m)
'October 30, 2025	7.3	

CORE DRILLING RECORD

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

Log of Borehole BH25-03

Project No: OTT-24014796-A0

Project: Proposed Residential Development

Location: 85 Gemini Way (Lot B), Ottawa, Ontario

Figure No. 8

Page. 1 of 1

Date Drilled: 'October 15, 2025

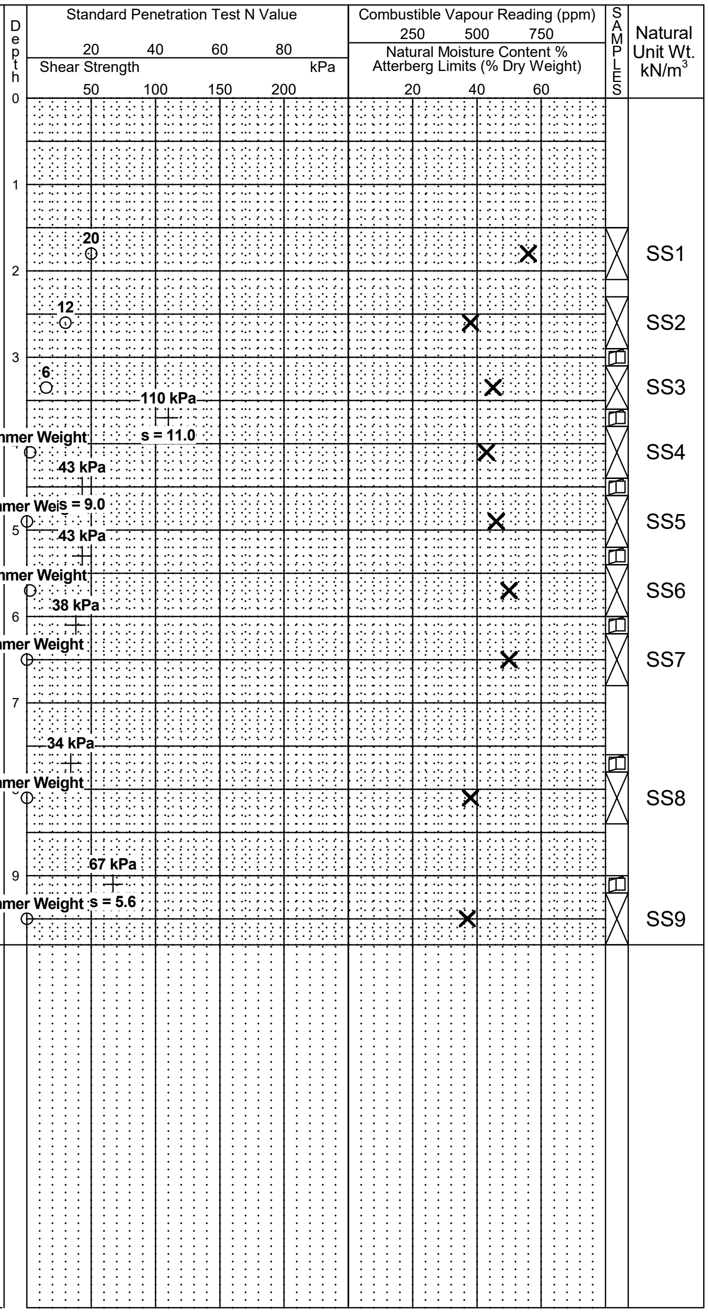
Drill Type: CME 75 Truck-Mounted Drill Rig

Datum: Geodetic Elevation

Logged by: SA   Checked by: SMP

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

Geodetic Elevation m	SOIL DESCRIPTION
86.59	**INFERRED FILL** Borehole advanced by power augering techniques (unsampled) from ground surface to 1.5 m depth.
85.1	**FILL** Silty sand and gravel, silty clay, roots and rootlets, brown, (compact)
84.7	**SILTY CLAY** With silty sand layers, brown, moist to wet, (very stiff)
82.8	**SILTY CLAY** With silty sand layers, grey, wet, (firm to stiff)
76.8	**Borehole Terminated at 9.8 m Depth**



NOTES:
1. Borehole data requires interpretation by EXP before use by others
2. Borehole backfilled upon completion of drilling.
3. Field work supervised by an EXP representative.
4. See Notes on Sample Descriptions
5. Log to be read with EXP Report OTT-24014796-A0

WATER LEVEL RECORDS

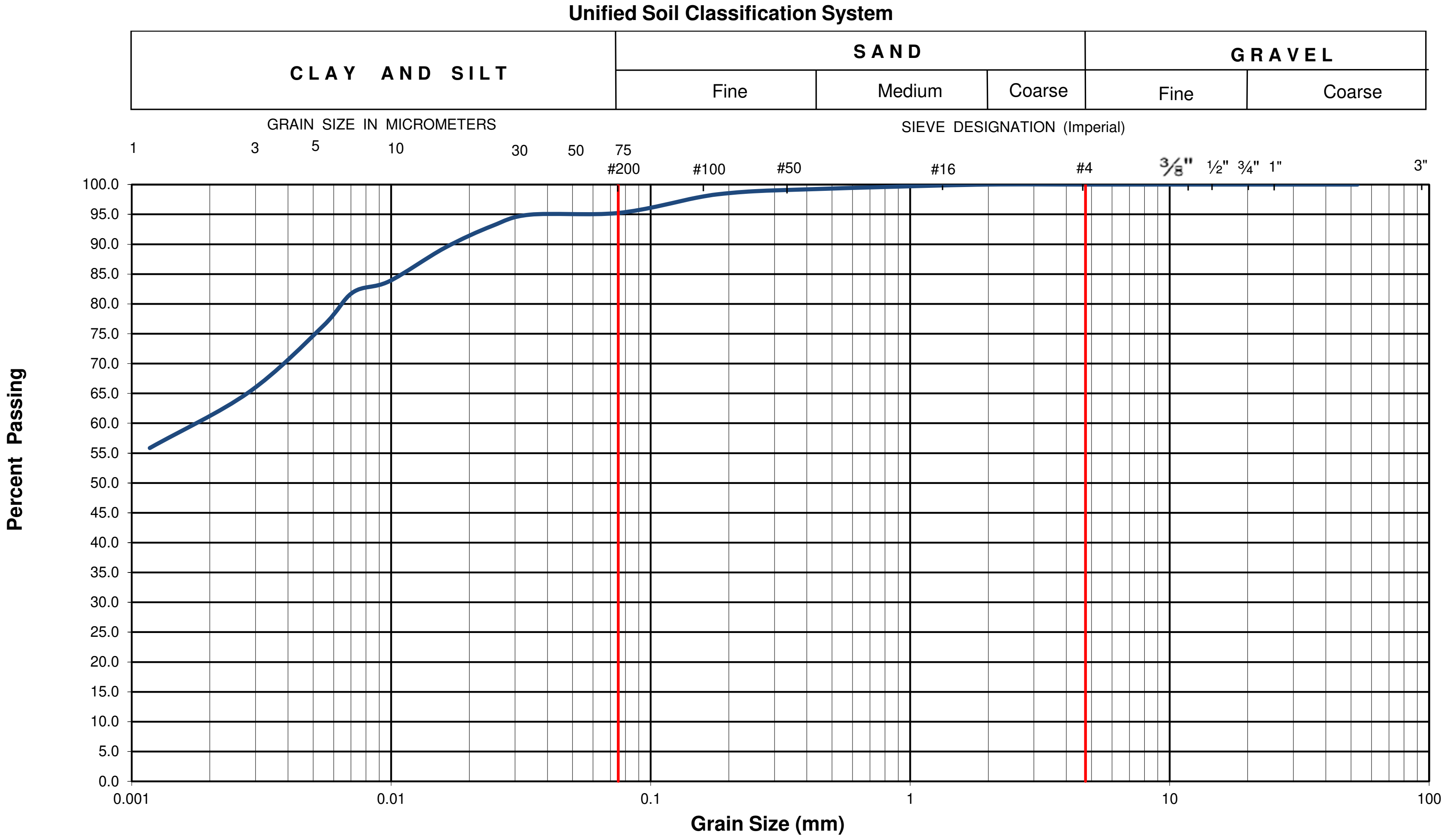
Date	Water Level (m)	Hole Open To (m)

CORE DRILLING RECORD

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

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

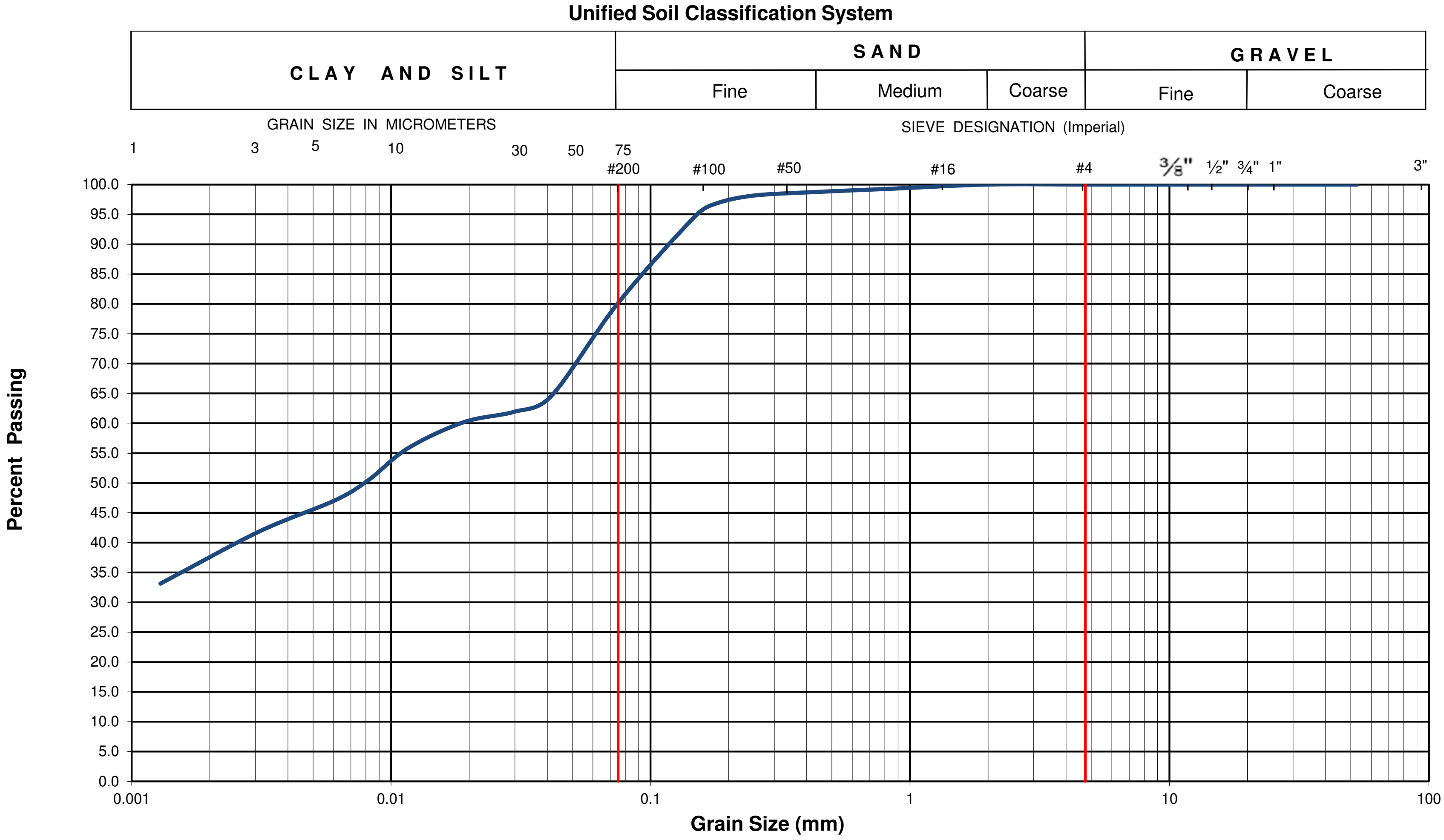
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.: OTT-24014796-A0	**Project Name :** Proposed Residential Development					
Client : Centurion Appelt (1 Centrepoint) LP	**Project Location :** 85 Gemini Way (Lot B), Ottawa, Ontario					
Date Sampled : December 12, 2024	**Borehole No:** BH24-01		**Sample No.:** SS4		**Depth (m) :** 2.3-2.9	
Sample Description :	**% Silt and Clay**	95	**% Sand** 5	**% Gravel**	0	**Figure :** 9
Sample Description :	**SILTY CLAY of HIGH PLASTICITY (CH): Trace Sand**					

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON   K2B 8H6

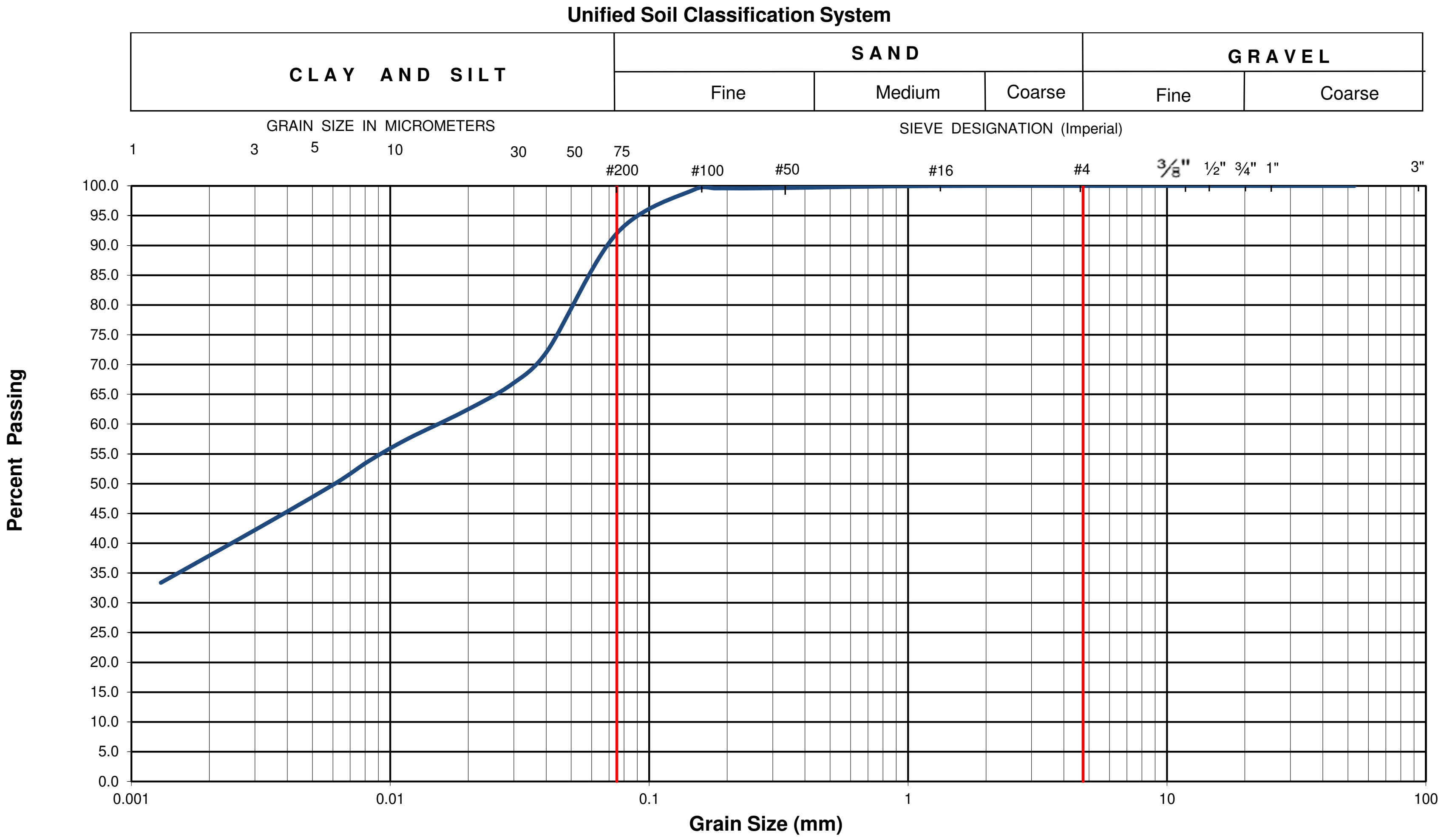
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-24014796-A0**	**Project Name :**	**Proposed Residential Development**				
Client :	**Centurion Appelt (1 Centrepoint) LP**	**Project Location :**	**85 Gemini Way (Lot B), Ottawa, Ontario**				
Date Sampled :	**December 12, 2024**	**Borehole No:**	**BH24-01**	**Sample No.:**	**SS9**	**Depth (m) :**	**7.6-8.2**
Sample Description :		**% Silt and Clay**	**80**	**% Sand**	**20**	**% Gravel**	**0**
Sample Description :	**SILTY CLAY of MEDIUM PLASTICITY (CI): Sandy**					**Figure :**	**10**

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON   K2B 8H6

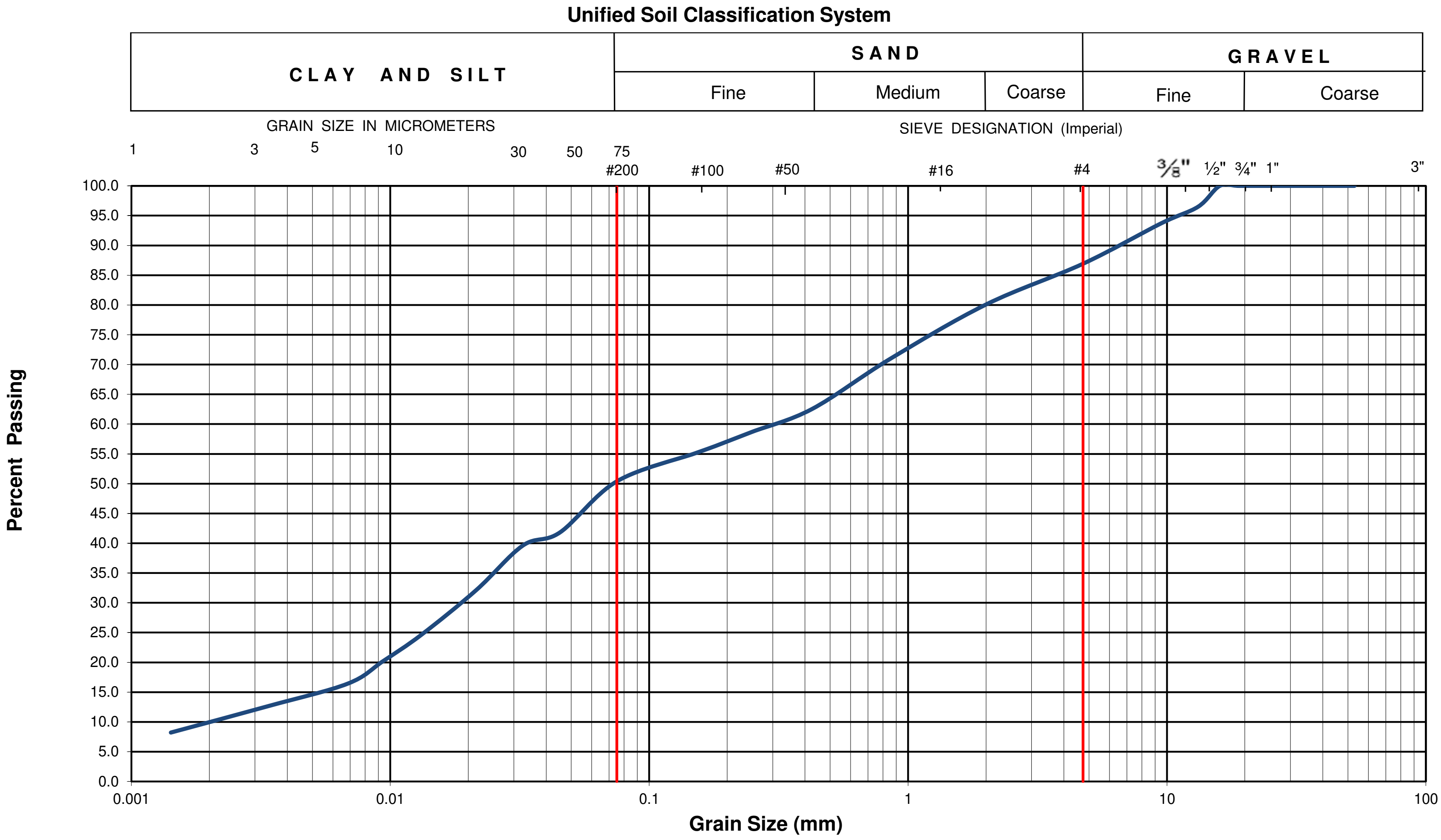
Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-24014796-A0**	**Project Name :**	**Proposed Residential Development**						
Client :	**Centurion Appelt (1 Centrepoint) LP**	**Project Location :**	**85 Gemini Way (Lot B), Ottawa, Ontario**						
Date Sampled :	**December 19, 2024**	**Borehole No:**	**BH24-03**	**Sample No.:**	**SS6**			**Depth (m) :**	**4.6-5.2**
Sample Description :		**% Silt and Clay**	**92**	**% Sand**	**8**	**% Gravel**	**0**	**Figure :**	**11**
Sample Description :	**SILTY CLAY of MEDIUM PLASTICITY (CI): Trace Sand**								

EXP Services Inc.
100-2650 Queensview Drive
Ottawa, ON K2B 8H6

Grain-Size Distribution Curve
Method of Test For Particle Size Analysis of Soil
ASTM C-136/ASTM D422



EXP Project No.:	**OTT-24014796-A0**	**Project Name :**	**Proposed Residential Development**						
Client :	**Centurion Appelt (1 Centrepoint) LP**	**Project Location :**	**85 Gemini Way (Lot B), Ottawa, Ontario**						
Date Sampled :	**December 12, 2024**	**Borehole No:**	**BH24-01**	**Sample No.:**	**SS11**			**Depth (m) :**	**10.7-11.3**
Sample Description :		**% Silt and Clay**	**50**	**% Sand**	**37**	**% Gravel**	**13**	**Figure :**	**12**
Sample Description :	**GLACIAL TILL: Silty Sand (SM) - Some Gravel and Clay**								

Run 1 - 12.5 to 13.9 m

Run 2 and Run 3 - 13.9 m to 15.8 m

Borehole No:	Core Runs/Depth	project	Project N0:
BH 24-03	Run 1 : 12.5 m - 13.9 m Run 2: 13.9 m - 14.7 m Run 3 : 14.7 m - 15.8 m	Proposed Residential Development 85 Gemini Way (Lot B), Ottawa, Ontario	OTT-24014796-A0
Date Cored Dec 20, 2024		Bedrock Core Photographs	FIGURE: 13

Appendix A: Seismic Shear Wave Velocity Sounding Survey Report by GPR

GEOPHYSICS GPR
INTERNATIONAL INC.

October 29th, 2025

Transmitted by email : ismail.taki@exp.com
Our ref : GPR25-06596-e

Mr. Ismail Taki, M.Eng., P.Eng.
Senior Manager, Earth & Environment, Eastern Region
exp Services inc.
Suite 100 - 2650 Queensview Drive
Ottawa ON K2B 8H6

Subject: **Shear Wave Velocity Sounding for the Site Designation Determination**
85 Gemini Way, Nepean, Ottawa (ON)

[Project #: OTT-24014796-A0]

Dear Mr. Taki,

Geophysics GPR International inc. has been mandated by **exp** Services inc. to carry out a seismic survey at 85 Gemini Way, Nepean, in Ottawa (ON). The geophysical investigation used the Multi-channel Analysis of Surface Waves (MASW) with the Spatial AutoCorrelation (SPAC), and the seismic refraction method. From the subsequent results, the seismic shear wave velocity values were calculated for the soils, to determine the Site Designation.

The surveys were conducted September 25th, 2025, by Mrs. Karyne Faguy, B.Sc. geophysics and Mr. Timothy Ward, tech. Figure 1 shows the regional location of the site and Figure 2 illustrates the location of the seismic spreads. Both figures are presented in the Appendix.

The following paragraphs briefly describe the principles of the testing method, the survey design, and the results presented in table and graph.

MASW Principle

The *Multi-channel Analysis of Surface Waves* (MASW) and the *SPatial AutoCorrelation* (SPAC or MAM for *Microtremors Array Method*) are seismic methods used to evaluate the shear wave velocities of subsurface materials through the analysis of the dispersion properties of the Rayleigh surface wave. The MASW is considered an "active" method, as the seismic signal is induced at known location and time in the geophones' spread axis. Conversely, the SPAC is considered a "passive" method, using the low frequency "signals" produced far away. The method can also be used with "active" seismic source records. The SPAC method generally allows deeper V_S soundings. Its dispersion curve can then be merged with the one of higher frequency from the MASW to calculate a more complete inversion. The dispersion properties are expressed as a change of velocities with respect to frequencies. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. The inversion of the Rayleigh wave dispersion curve yields a shear wave (V_S) velocity depth profile (sounding).

Figure 3 schematically outlines the basic operating procedure for the MASW method. Figure 4 illustrates an example of one of the MASW/SPAC records, a corresponding spectrogram analysis and resulting 1D V_S model.

INTERPRETATION

The main processing sequence involved data inspection and edition when required; spectral analysis (from MASW and SPAC); picking the fundamental mode; and 1D inversion of the MASW and SPAC shot records using the SeisImagerSW™ software. The data inversions used a nonlinear least squares algorithm.

In theory, all the shot records for a given seismic spread should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation, local surface seismic velocities variations, and/or dipping of overburden layers or rock. In general, the precision of the calculated seismic shear wave velocities (V_S) is around 15% or better.

More detailed descriptions of these methods are presented in *Shear Wave Velocity Measurement Guidelines for Canadian Seismic Site Characterization in Soil and Rock*, Hunter, J.A., Crow, H.L., et al., Geological Surveys of Canada, General Information Product 110, 2015.

SURVEY DESIGN

The seismic acquisition layouts were located beside an existing building, south of the intersection of Baseline Rd and Constellation Dr (Figure 2). The geophone spacing was 3.0 metres for the main seismic line, using 24 geophones. A shorter one with geophone spacing of 1.0 metre was dedicated to the near surface materials. The seismic records were produced with a Terraloc PRO seismograph (from ABEM), and the geophones were 4.5 Hz.

The seismic records counted 4096 data, sampled at 1000 µs for the MASW surveys, and at 40 µs for the seismic refraction ones. The records included a pre-trigged portion of 10 ms. A 7.25 kg sledgehammer was used as the energy source, with impacts being recorded off both ends of the seismic spreads. A stacking procedure was also used to improve the Signal / Noise ratio for the seismic records. The shear wave depth sounding can be considered as the average of the bulk area within the geophone spread, especially for its central half-length.

RESULTS

The rock depth was calculated between 12 and 14.5 metres deep from seismic refraction (± 1 metre). Its seismic velocity (V_S) was calculated at 1825 m/s. These parameters were used for the initial geophysical models, prior to the inversions of the MASW analysis results.

The MASW calculated V_S results are illustrated at Figure 5. Some low seismic velocities were calculated from 1 to 9 metres deep.

The $\overline{V}_{S30}$ value results from the harmonic average of the shear wave velocities, from the surface to 30 metres deep. It is calculated by dividing the total depth of interest (30 metres) by the sum of the time spent in each velocity layer from the surface down to 30 metres, as:

$$\bar{V}_{S30} = \frac{\sum_{i=1}^{N} \mathrm{H}_i}{\sum_{i=1}^{N} {}^{H_i}/_{V_i}} \quad | \quad \sum_{i=1}^{N} \mathrm{H}_i \; = \; 30 \text{ m}$$

(N: number of layers; H_i : thickness of layer "i" ; V_i : V_S of layer "i")

Thus, the $\overline{V}_{S30}$ value represents the seismic shear wave velocity of an equivalent homogeneous single layer response, between the surface and 30 metres deep.

The calculation of the $\overline{V}_{S30}$ value is presented at Table 1. The Site Designation is X_{389}, which corresponds to the Site Class "C".

CONCLUSION

Geophysical surveys were carried out to identify the Site Designation at 85 Gemini Way, Nepean, in Ottawa (ON). The seismic surveys used the MASW and the SPAC analysis to calculate the $\overline{V}_{S30}$ value. Its calculation is presented at Table 1.

For the actual site, the Site Designation is X_{389}, corresponding to the Site Class "C" ($360 < \overline{V}_{S30} \leq 760$ m/s), as determined through the MASW and SPAC methods, Table 4.1.8.4.-B of the NBC (2020), and the Building Code, O. Reg. 163/24.

It must be noted that some low seismic velocities were calculated from 1 to 9 metres deep. A geotechnical assessment of the corresponding materials could be required for the potential of liquefaction, the degree of sensitivity of the clay and other critical parameters.

It must also be noted that other geotechnical information gleaned on site; including the presence of liquefiable soils, very soft clays, high moisture content etc. (cf. Tables 4.1.8.4.-A and 4.1.8.4.-B of the NBC 2020) can supersede the Site Classification and the Site Designation provided in this report based on the $\overline{V}_{S30}$ value.

The V_S values calculated are representative of the in situ materials and are not corrected for the total and effective stresses.

Hoping the whole to your satisfaction, we remain yours truly,

Jean-Luc Arsenault, M.A.Sc., P.Eng.
Senior Project Manager

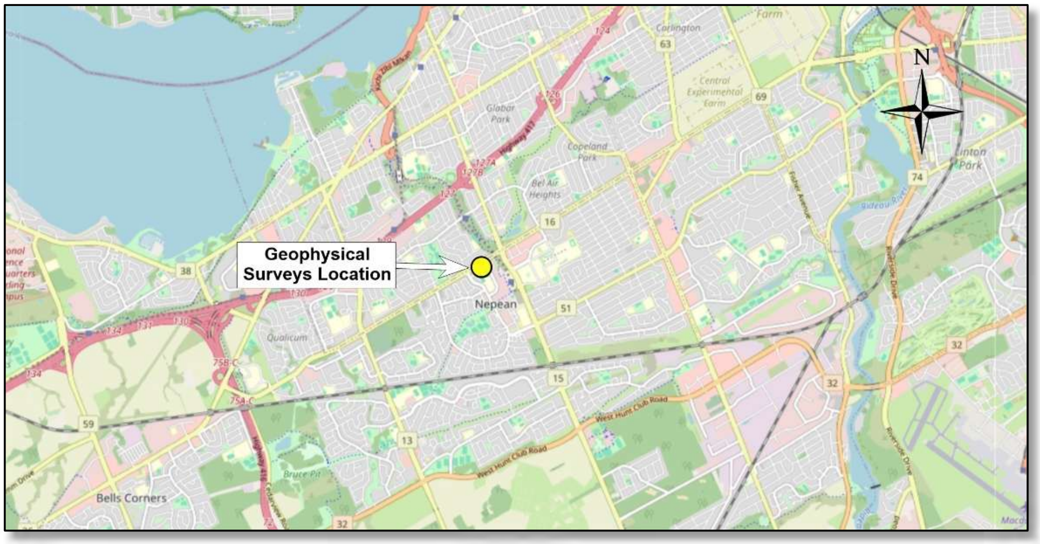


Figure 1: Regional location of the Site
(Source : OpenStreetMap©)

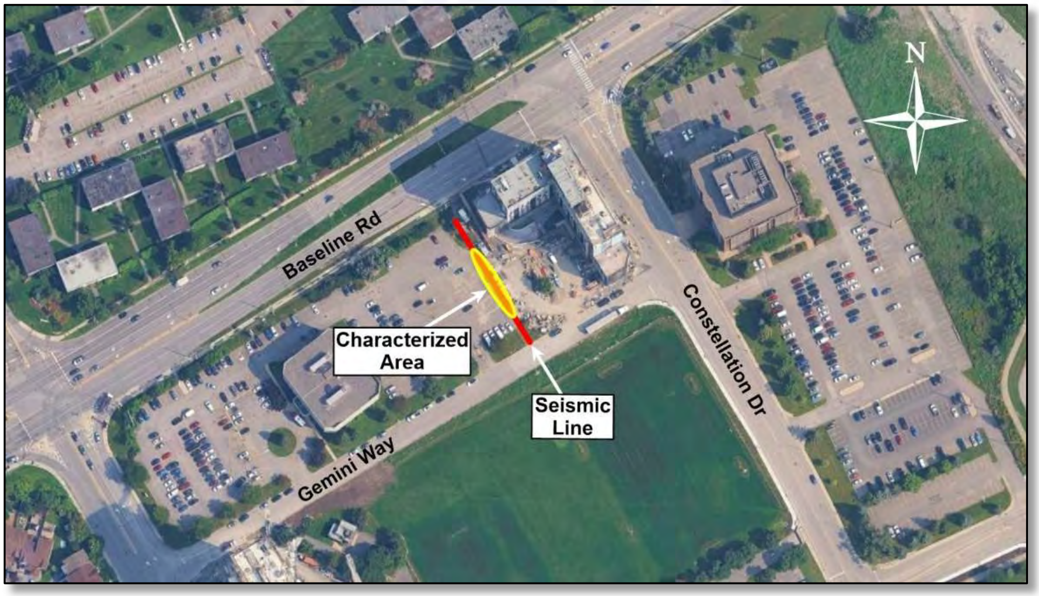


Figure 2: Location of the seismic spreads
(source: *Google Earth™*)

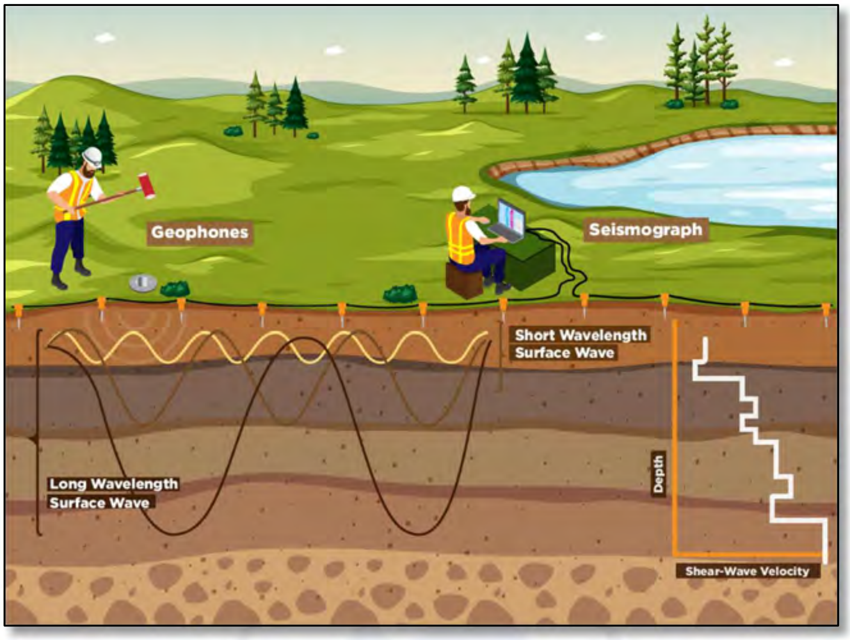


Figure 3: MASW Operating Principle

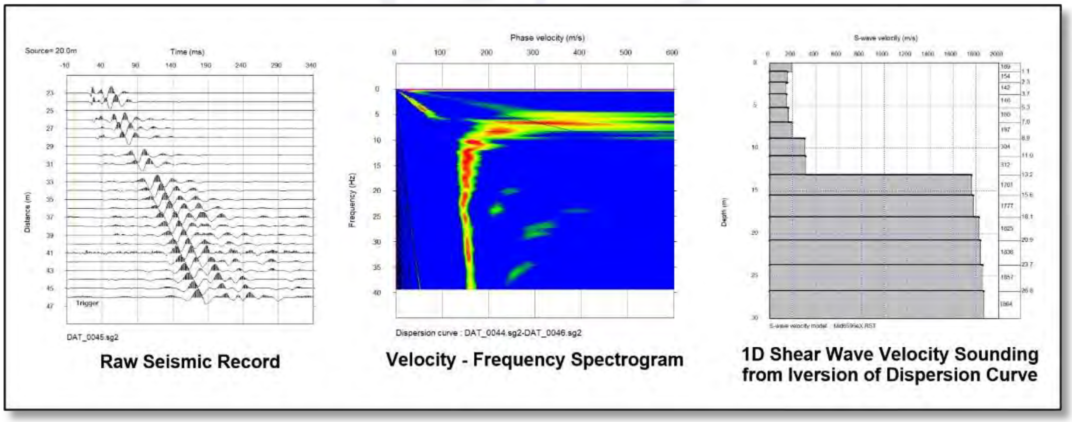


Figure 4: Example of a MASW/SPAC record, Phase Velocity - Frequency curve of the Rayleigh wave and resulting 1D Shear Wave Velocity Model

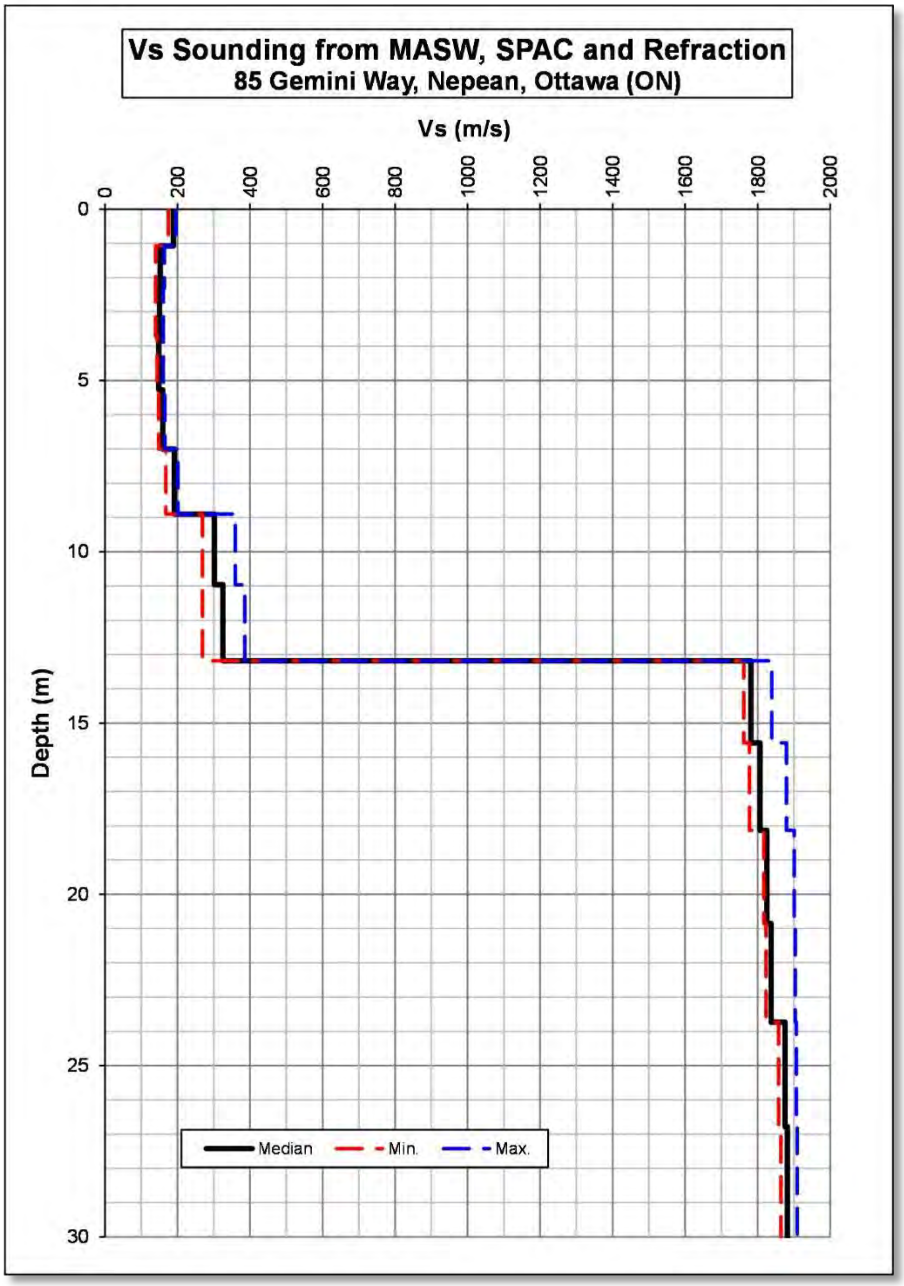


Figure 5: MASW Shear-Wave Velocity Sounding

TABLE 1

$\bar{V}_{S30}$ Calculation for the Site Designation (actual site)

Depth	Vs			Thickness	Cumulative Thickness	Delay for med. Vs	Cumulative Delay	Vs at given Depth
	Min.	Median	Max.					
(m)	(m/s)	(m/s)	(m/s)	(m)	(m)	(s)	(s)	(m/s)
0	**174.6**	188.6	195.5	**Grade Level (September 25th, 2025)**				
1.07	**139.9**	**152.5**	**164.1**	1.07	1.07	0.005682	0.005682	188.6
2.31	**139.5**	**150.4**	**160.6**	1.24	2.31	0.008107	0.013788	167.4
3.71	**143.9**	**148.5**	**160.5**	1.40	3.71	0.009316	0.023104	160.5
5.27	**147.8**	**159.7**	**165.1**	1.57	5.27	0.010546	0.033650	156.8
7.01	**168.1**	191.4	201.5	1.73	7.01	0.010840	0.044490	157.5
8.90	268.7	301.7	359.5	1.90	8.90	0.009901	0.054391	163.6
10.96	268.9	325.2	385.9	2.06	10.96	0.006830	0.061222	179.0
13.19	1761.6	1781.3	1839.1	2.23	13.19	0.006844	0.068065	193.7
15.58	1777.3	1805.9	1879.1	2.39	15.58	0.001342	0.069407	224.4
18.13	1817.2	1825.7	1900.7	2.55	18.13	0.001415	0.070822	256.0
20.85	1823.0	1836.8	1903.7	2.72	20.85	0.001490	0.072312	288.4
23.74	1857.7	1875.3	1906.5	2.88	23.74	0.001570	0.073882	321.3
26.79	1864.5	1881.5	1909.3	3.05	26.79	0.001626	0.075508	354.7
30				3.21	30.00	0.001708	0.077216	388.5

V_{S30} (m/s)	**388.5**
Class	**C (1)**

(1) Some low seismic velocities were calculated from 1 to 9 metres deep. A geotechnical assessment of the corresponding materials could be required.

Appendix B - Laboratory Certificate of Analysis Report by AGAT

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
http://www.agatlabs.com

CLIENT NAME: EXP SERVICES INC
2650 QUEENSVIEW DRIVE, UNIT 100
OTTAWA, ON K2B8H6
(613) 688-1899
ATTENTION TO: Ismail M. Taki
PROJECT: OTT-24014795-A0
AGAT WORK ORDER: 25Z236572
SOIL ANALYSIS REVIEWED BY: Nivine Basily, Inorganic Team Lead
DATE REPORTED: Jan 14, 2025
PAGES (INCLUDING COVER): 5
VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*Notes

Disclaimer:

- *All work conducted herein has been done using accepted standard protocols, and generally accepted practices and methods. AGAT test methods may incorporate modifications from the specified reference methods to improve performance.*
- *All samples will be disposed of within 30 days after receipt unless a Long Term Storage Agreement is signed and returned. Some specialty analysis may be exempt, please contact your Client Project Manager for details.*
- *AGAT's liability in connection with any delay, performance or non-performance of these services is only to the Client and does not extend to any other third party. Unless expressly agreed otherwise in writing, AGAT's liability is limited to the actual cost of the specific analysis or analyses included in the services.*
- *This Certificate shall not be reproduced except in full, without the written approval of the laboratory.*
- *The test results reported herewith relate only to the samples as received by the laboratory.*
- *Application of guidelines is provided "as is" without warranty of any kind, either expressed or implied, including, but not limited to, warranties of merchantability, fitness for a particular purpose, or non-infringement. AGAT assumes no responsibility for any errors or omissions in the guidelines contained in this document.*
- *All reportable information is available on request from AGAT Laboratories, in accordance with ISO/IEC 17025:2017, ISO/IEC 17025:2005 (Quebec), DR-12-PALA and/or NELAP Standards.*
- *This document is signed by an authorized signatory who meets the requirements of the MELCCFP, CALA, CCN and NELAP.*
- *For environmental samples in the Province of Quebec: The analysis is performed on and results apply to samples as received. A temperature above 6°C upon receipt, as indicated in the Sample Reception Notification (SRN), could indicate the integrity of the samples has been compromised if the delay between sampling and submission to the laboratory could not be minimized.*

Member of: Association of Professional Engineers and Geoscientists of Alberta (APEGA)
Western Enviro-Agricultural Laboratory Association (WEALA)
Environmental Services Association of Alberta (ESAA)

AGAT Laboratories is accredited to ISO/IEC 17025 by the Canadian Association for Laboratory Accreditation Inc. (CALA) and/or Standards Council of Canada (SCC) for specific tests listed on the scope of accreditation. AGAT Laboratories (Mississauga) is also accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) for specific drinking water tests. Accreditations are location and parameter specific. A complete listing of parameters for each location is available from www.cala.ca and/or www.scc.ca. The tests in this report may not necessarily be included in the scope of accreditation. Measurement Uncertainty is not taken into consideration when stating conformity with a specified requirement.

Certificate of Analysis

AGAT WORK ORDER: 25Z236572

PROJECT: OTT-24014795-A0

CLIENT NAME: EXP SERVICES INC

ATTENTION TO: Ismail M. Taki

SAMPLING SITE:

SAMPLED BY:

(Soil) Inorganic Chemistry

DATE RECEIVED: 2025-01-06

DATE REPORTED: 2025-01-14

			SAMPLE DESCRIPTION:	BH24-2 SS7 15'-17'	BH24-2 SS11 35'-37'
			SAMPLE TYPE:	Soil	Soil
			DATE SAMPLED:	2024-12-12	2024-12-12
Parameter	**Unit**	**G / S**	**RDL**	**6446832**	**6446843**
Chloride (2:1)	µg/g		2	532	838
Sulphate (2:1)	µg/g		2	123	209
pH (2:1)	pH Units		NA	7.63	8.14
Electrical Conductivity (2:1)	mS/cm		0.005	1.21	1.41

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

6446832-6446843 pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter.

Analysis performed at AGAT Toronto (unless marked by *)

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO
CHARTERED
NIVINE BASILY
CHEMIST

Certified By:

Quality Assurance

CLIENT NAME: EXP SERVICES INC

PROJECT: OTT-24014795-A0

SAMPLING SITE:

AGAT WORK ORDER: 25Z236572

ATTENTION TO: Ismail M. Taki

SAMPLED BY:

Soil Analysis

RPT Date: Jan 14, 2025			DUPLICATE			Method Blank	REFERENCE MATERIAL			METHOD BLANK SPIKE			MATRIX SPIKE		
PARAMETER	Batch	Sample Id	Dup #1	Dup #2	RPD		Measured Value	Acceptable Limits		Recovery	Acceptable Limits		Recovery	Acceptable Limits	
								Lower	Upper		Lower	Upper		Lower	Upper
(Soil) Inorganic Chemistry															
Chloride (2:1)	6446832	6446832	532	550	3.3%	< 2	95%	70%	130%	98%	80%	120%	NA	70%	130%
Sulphate (2:1)	6446832	6446832	123	124	0.8%	< 2	100%	70%	130%	101%	80%	120%	97%	70%	130%
pH (2:1)	6446832	6446832	7.63	7.96	4.2%	NA	89%	80%	120%						
Electrical Conductivity (2:1)	6446832	6446832	1.21	1.11	8.6%	< 0.005	96%	80%	120%						

Comments: NA signifies Not Applicable.
pH duplicates QA acceptance criteria was met relative as stated in Table 5-15 of Analytical Protocol document.

Duplicate NA: results are under 5X the RDL and will not be calculated.

Matrix spike NA: Spike level < native concentration. Matrix spike acceptance limits do not apply and are not calculated.

NIVINE BASILY

Certified By:

Method Summary

CLIENT NAME: EXP SERVICES INC
PROJECT: OTT-24014795-A0
SAMPLING SITE:

AGAT WORK ORDER: 25Z236572
ATTENTION TO: Ismail M. Taki
SAMPLED BY:

PARAMETER	AGAT S.O.P	LITERATURE REFERENCE	ANALYTICAL TECHNIQUE
Soil Analysis			
Chloride (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
Sulphate (2:1)	INOR-93-6004	modified from SM 4110 B	ION CHROMATOGRAPH
pH (2:1)	INOR 93-6031	modified from EPA 9045D and MCKEAGUE 3.11	PH METER
Electrical Conductivity (2:1)	INOR-93-6075	modified from MSA PART 3, CH 14 and SM 2510 B	PC TITRATE

AGAT Laboratories

Have feedback?
Scan here for a quick survey!

5835 Coopers Avenue
Mississauga, Ontario L4Z 1Y2
Ph: 905.712.5100 Fax: 905.712.5122
webearth.agatlabs.com

Chain of Custody Record

If this is a Drinking Water sample, please use Drinking Water Chain of Custody Form (potable water consumed by humans)

Laboratory Use Only

Work Order #: 25Z236572

Cooler Quantity: n/a - bag

Arrival Temperatures: 19.0 | 18.9 | 18.8

~~Depot~~ Temperatures: 4.5 | 4.3 | 1.2

Custody Seal Intact: ☐Yes ☐No ☐N/A

Notes: L / I

Report Information:

Company: EXP Services
Contact: Ismail Taki
Address: 2650 Queensview Drive, Suit 100
Ottawa, Ontario, K2B 8H6
Phone: 613-688-1899 Fax:
Reports to be sent to:
1. Email: Ismail.taki@exp.com
2. Email: ryan.digiuseppe@exp.com

Regulatory Requirements:

(Please check all applicable boxes)

☐ Regulation 153/04
Table ____ *Indicate One*
☐Ind/Com
☐Res/Park
☐Agriculture
Soil Texture *(Check One)*
☐Coarse
☐Fine

☐ Regulation 406
Table ____ *Indicate One*
☐Ind/Com
☐Res/Park
☐Agriculture
☐ Regulation 558
☐ CCME

☐ Sewer Use
☐Sanitary ☐Storm
Region ____
☐ Prov. Water Quality Objectives (PWQO)
☐ Other
____ *Indicate One*

Turnaround Time (TAT) Required:

Regular TAT ☑ 5 to 7 Business Days

Rush TAT (Rush Surcharges Apply)

☐ 3 Business Days ☐ 2 Business Days ☐ Next Business Day

OR Date Required (Rush Surcharges May Apply): ____

Please provide prior notification for rush TAT
*TAT is exclusive of weekends and statutory holidays

For 'Same Day' analysis, please contact your AGAT CSR

Project Information:

Project: OTT-24014795-A0
Site Location: 1 Centrepointe Drive, Ottawa
Sampled By: EXP
AGAT Quote #: ____ PO: ____
Please note: If quotation number is not provided, client will be billed full price for analysis.

Is this submission for a **Record of Site Condition (RSC)**?
☐ Yes ☐ No

Report Guideline on *Certificate of Analysis*
☐ Yes ☐ No

Legal Sample ☐

Invoice Information:

Bill To Same: Yes ☑ No ☐

Company:
Contact:
Address:
Email:

Sample Matrix Legend

GW	Ground Water	SD	Sediment
O	Oil	SW	Surface Water
P	Paint	R	Rock/Shale
S	Soil		

Sample Identification	Date Sampled	Time Sampled	# of Containers	Sample Matrix	Comments/ Special Instructions	Field Filtered - Metals, Hg, CrVI, DOC Y / N	O. Reg 153: Metals & Inorganics	O. Reg 153: Metals - ☐ CrVI, ☐ Hg, ☐ HWSB	O. Reg 153: BTEX, F1-F4 PHCs	VOC	PAHs	PCBs: Aroclors ☐	O. Reg 406: Regulation 406 Characterization Package pH, Metals, BTEX, F1-F4	O. Reg 406: EC, SAR	O. Reg 406: Regulation 406 SPLP Rainwater Leach mSPLP: ☐ Metals ☐ VOCs ☐ SVOCs ☐ OC	O. Reg 558: Landfill Disposal Characterization TCLP: TCLP: ☐ M&I ☐ VOCs ☐ ABNs ☐ B(a)P ☐ PCBs	Corrosivity: ☐ Moisture ☐ Sulphide	pH	Sulphates	Chlorides	Electro Conductivity			Potentially Hazardous or High Concentration (Y/N)
1. BH24-2 SS7 15'-17'	Dec 12	AM PM	1															☑	☑	☑	☑			
2. BH24-2 SS11 35'-37'	Dec 12	AM PM	1															☑	☑	☑	☑			
3.		AM PM																						
4.		AM PM																						
5.		AM PM																						
6.		AM PM																						
7.		AM PM																						
8.		AM PM																						
9.		AM PM																						
10.		AM PM																						
11.		AM PM																						

Samples Relinquished By (Print Name and Sign):	Date	Time	Samples Received By (Print Name and Sign):	Date	Time	
				01/06/25	16h40	
CA TO Dewo	01/07/25	1500		Jan 8	9:15h	Page ____ of ____
						N°:

Pink Copy - Client | Yellow Copy - AGAT | White Copy- AGAT

Legal Notification

This report was prepared by EXP Services Inc. for the account of Centurion Appelt (1 Centrepoint) LP.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP Services Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project.

List of Distribution

Report Distributed To:

Joshua Saltzman jsaltzman@Appeltproperties.com