



FINAL REPORT

Geotechnical Investigation

*Proposed Residential Buildings
30 Cleary Avenue, Ottawa, ON*

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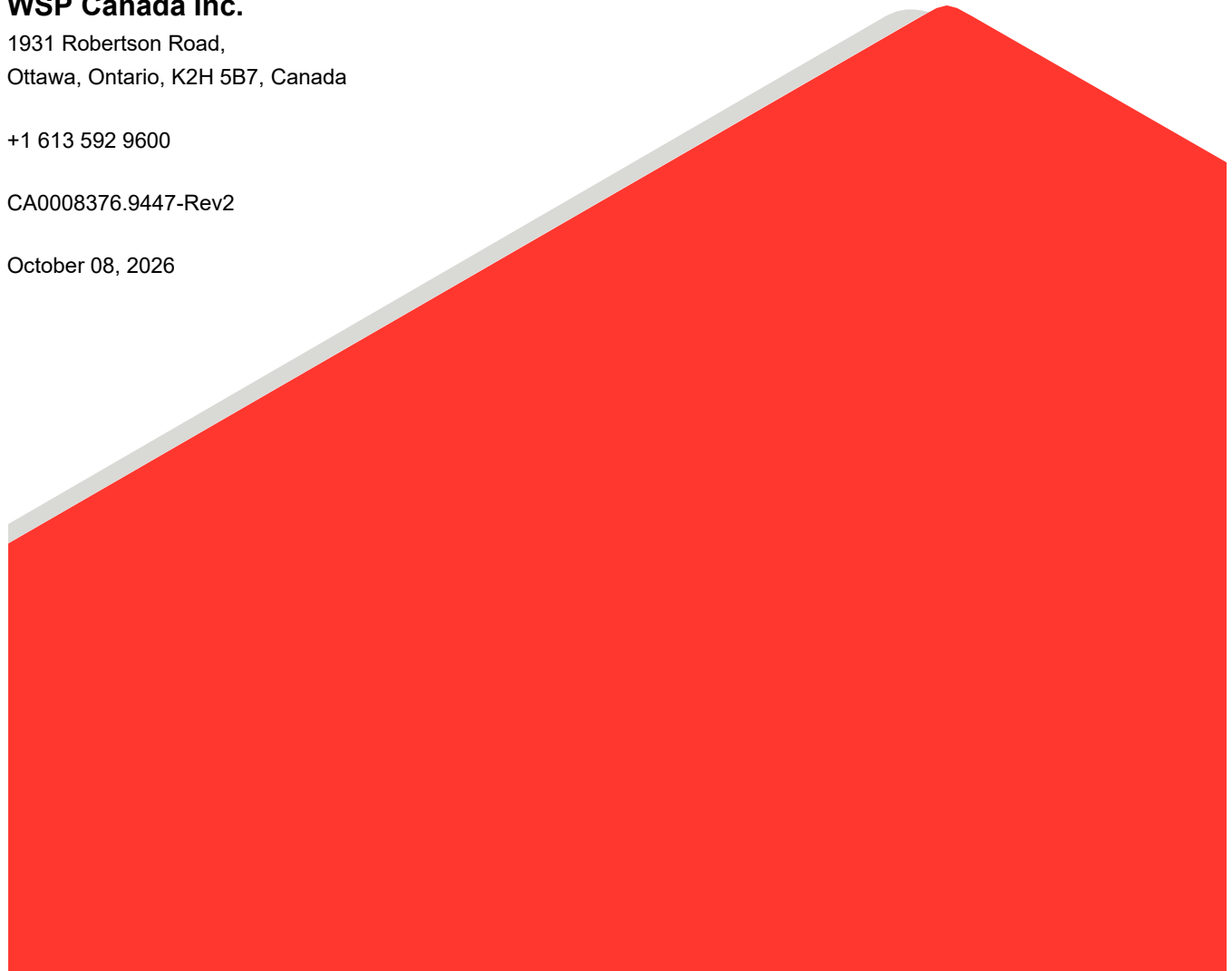
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the Site of two proposed residential buildings to be located at 30 Cleary Avenue, Ottawa, Ontario.

The purpose of this geotechnical investigation was to assess the general subsurface conditions at the site by means of a limited number of boreholes. Based on an interpretation of the factual information obtained, a general description of the subsurface conditions is presented. These interpreted subsurface conditions and available project details were used to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

An environmental Site Assessment was completed at the same time as the present geotechnical investigation. The results of this assessment are provided in a separate report.

A geophysical survey consisting of Multichannel Analysis of Surface-Waves (MASW) was completed by WSP at the project site. The factual data obtained are presented in a technical memorandum in Appendix G, titled "*MASW Survey Results – 30 Cleary Ave Ottawa*", and dated April 27, 2026.

The reader is referred to the "*Important Information and Limitations of This Report*" which follows the text but forms an integral part of this document.

2.0 DESCRIPTION OF PROJECT AND SITE

The site of the proposed development is located at 30 Cleary Avenue, Ottawa, Ontario. The approximate location of the site is shown on the Key Map inset on the attached Site Plan (Figure 1).

The proposed Site has not been previously developed but is currently surrounded by a number of structures:

- The proposed development area measures approximately 100 m by 20 m in plan view and is currently covered by various landscaping and surface parking areas.
- The proposed residential buildings will be located in the northwest corner of the larger overall site, to the north of the existing river Parkway Children's Centre and southwest of the existing First Unitarian Congregation Church. The proposed building footprints are shown on the Site plan, Figure 1
- The proposed development will include a high-rise residential building of up to 16 storeys with two levels of underground parking at an approximate depth of 7.0 m below ground surface, and a mid-rise residential building of up to 6 storeys with two levels of underground parking at an approximate depth of 6.4 m below ground surface.
- Details of the exact floor elevations were provided to WSP at the time of preparing this report. It is understood that no significant regrading of the site will be required to accommodate the development.

2.1 Available Subsurface Information

Previous subsurface investigations at or near the site were carried out by Golder Associates Ltd, and McRostie & Associates Ltd. The locations of those previous boreholes are shown on the attached Site Plan (Figure 1). The following reports were reviewed in the assessment of site conditions for this study, which include the investigations for the existing development:

- 1) Report to Charlesfort Developments by Golder Associated Ltd. titled "Phase II Environmental Site Assessment and Remedial Action Plan 761 and 763 Richmond Road, Ottawa, Ontario" dated August 2007 (Report No. 04-1120-806-1).
- 2) Report to Unitarian Church by McRostie & Associates Ltd. titled "Preliminary Report on Subsurface Investigation for Unitarian Church, Leafloor Avenue, Ottawa, Ontario" dated May 19, 1964 (Report No. SF-763).
- 3) Report to Craig & Kohler, Architects and Robert Halsall and Associates, Structural Engineers titled "Report on Foundation Investigation for Proposed Unitarian Church, Leafloor Avenue, Ottawa, Ontario" dated October 22, 1965 (Report No. SF-917).

Based on the available information, the subsurface conditions are anticipated to consist of surficial fill material over native sand, underlain by glacial till. The bedrock surface was found to vary from about 0.4 to 3.3 m below the existing ground surface in the previous investigations in the general area.

Published bedrock geology mapping indicates that the site is underlain by limestone with shale interbeds of the Gull River Formation.

3.0 PROCEDURE

The field work for the current geotechnical and environmental assessment was carried out between August 24 and September 29, 2023, and on April 15, 2026. During that time, a total of twelve boreholes (numbered BH23-01 to BH23-07, BH23-07A to BH23-09, BH26-01 and BH26-02) were advanced at the approximate locations shown in the site plan in Figure 1.

The boreholes were advanced with a truck-mounted hollow stem auger drill rig supplied and operated by George Downing Estate Drilling Ltd. of Grenville, Quebec.

Standard Penetration Tests (SPTs) were carried out within the overburden at various intervals of depth in general conformance with ASTM D 1586. Soil samples were recovered using split-spoon sampling equipment.

Nine boreholes were advanced to refusal on the bedrock surface at depths ranging from 0.8 to 2.4 m. Upon encountering refusal, boreholes BH23-01, BH23-05, and BH23-07A were advanced into the bedrock using rotary diamond drilling techniques while retrieving NQ sized core up to a depth of approximately 9.9 m. HQ sized core were retrieved in boreholes BH26-01 and BH26-02 to a depth of approximately 11.9 m.

Monitoring wells were sealed into five boreholes to allow for subsequent measurements of stabilized groundwater levels and in-situ hydraulic conductivity testing. The monitoring wells consisted of 32 and 50 mm inside diameter rigid PVC pipe with 3 m long slotted screen sections, installed within silica sand backfill, and sealed by a section of bentonite hole plug. Measurement of groundwater levels was completed on September 05, 2023, October 12, 2023, May 01, 2026 and May 08, 2026.

The fieldwork was supervised by WSP staff who logged the boreholes, directed the in-situ testing, and collected the soil samples retrieved in the boreholes. The samples obtained during the fieldwork were brought to WSP's laboratory for further examination and laboratory testing.

The laboratory testing included determination of natural water content and grain size distribution on selected soil samples. Two samples of soil were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements.

The borehole locations were marked in the field and surveyed by WSP. The position and ground surface elevation at the borehole location were determined using a Trimble R8 GPS survey unit. The Geodetic reference system used for the survey is the North American Datum of 1983 (NAD83). The borehole coordinates are based on the Universal Transverse Mercator (UTM Zone 09) coordinate system. The elevations are referenced to the Geodetic datum (CGVD28).

4.0 SUBSURFACE STRATIGRAPHY

4.1 General

The soil descriptions provided in this report are based on accepted standard methods of classification and identification routinely used in current geotechnical state of practice. The method of soil classification used by WSP is attached in Appendix A.

The subsurface soil and groundwater conditions encountered in the borehole and the results of in situ and laboratory testing from the current investigation are given on the Record of Borehole sheets presented in Appendix A. The Record of Borehole sheets from the previous investigation at the site are provided for reference in Appendix B. The results of geotechnical laboratory testing from the current investigation are also presented in Appendix C. The results of basic soil and water chemical testing are included in Appendix D. Rock core photographs are provided in Appendix E. The hydraulic conductivity testing and estimated groundwater inflow are included in Appendix F and the geophysical survey results are presented in a technical memorandum in Appendix G.

The borehole locations from the current and previous investigations are shown on Figure 1.

In general, the subsurface stratigraphy encountered within the footprint of the proposed structures consists of surficial fill layer, underlain by native sand and silt deposit, over Glacial Till overlying limestone with thin shale interbeds.

4.2 Pavement Structure

A layer of asphaltic concrete, ranging from 30 to 130 mm thick, was encountered at BH23-01 to BH23-04, BH23-06, BH23-09 and BH26-02 (which were drilled in existing parking areas) during the current investigation.

4.3 Surficial Fill Materials

A thin layer of fill material was present underlying the asphaltic concrete. Within the footprint of the new development, the fill extended to depths of up to 0.86 m below the original ground surface.

The fill layer is brown to grey in colour, with measured SPT “N” values ranging from 6 to 71 blows per 0.3 m of penetration, indicating a loose to very dense state of packing.

The results of natural moisture content testing carried out on eight samples of the fill layer gave values ranging between 1 and 4 percent. The results of grain size distribution testing carried out on two samples of the fill are presented in appendix C.

As the proposed building footprint currently contains a below grade level, it is anticipated that the above noted materials will be entirely removed during construction of the proposed buildings.

4.4 Glacial Till

A deposit of glacial till was encountered beneath the fill in all boreholes. The glacial till was fully penetrated and proven to depths of 1.8 m below ground surface in all boreholes except borehole 23-07 in which the glacial till was proven to 2.4m. In general, the glacial till consists of a heterogeneous mixture of cobbles, boulders, clay and gravel in a matrix of silty sand.

Standard penetration tests carried out within this layer gave 'N' values ranging from 8 to 42 blows per 0.3 m of penetration, indicating a compact to dense state of packing.

The higher SPT N values may be due to the presence of cobbles and boulders and may not reflect the state of packing of the soil matrix.

The results of natural moisture content testing carried out on nine samples of the glacial till layer gave values ranging between 5 and 27 percent. The results of grain size distribution testing carried out on two samples of the till are presented in appendix C.

4.5 Bedrock / Auger Refusal

Refusal to augering within the footprint of the proposed structures was encountered in all boreholes during the current investigation at depths ranging from 0.9 to 2.4 m below the existing ground surface (0.9 m to 1.9 m in the area in which the proposed buildings will be located). The bedrock was cored in five of the current boreholes to a maximum depth of 10 m below the existing ground surface. The bedrock consists of grey limestone with shale interbeds up to approximately 8 m, underlain by sandstone bedrock to approximately 12 m below ground surface. The following table summarizes the ground surface, bedrock or auger refusal depths and elevations, and core lengths as encountered at the borehole locations within (or near to) the footprint of the proposed buildings:

Borehole/ Test Pit Number	Ground Surface Elevation (m)	Depth to Bedrock Surface or Auger Refusal (m)	Core Length (m)	Bedrock or Auger Refusal Elevation (m)
BH26-01 (WSP, 2026)	61.26	1.75	10.14	59.51
BH26-02 (WSP, 2026)	61.04	1.58	10.38	54.35
BH 23-01 (WSP,2023)	59.91	0.86	7.04	59.05
BH 23-02 (WSP,2023)	60.22	1.47	-	58.75
BH 23-03 (WSP,2023)	60.59	1.83	-	58.76
BH 23-04 (WSP,2023)	61.15	1.37	-	59.78
BH 23-05 (WSP,2023)	62.13	1.93	6.04	60.20
BH 23-06 (WSP,2023)	61.97	1.50	-	60.47
BH 23-07 (WSP,2023)	63.37	2.44	-	60.93
BH 23-07A (WSP, 2023)	63.37	4.47	5.47	53.43
BH 23-08 (WSP,2023)	61.73	1.30	-	60.43
BH 23-09 (WSP,2023)	59.68	1.87	-	57.81
No. 1 (Mcrostie,1964)	-	0.48	1.52	-
No. 2 (Mcrostie,1964)	-	3.53	1.55	-
No. 3 (Mcrostie,1964)	-	2.34	1.52	-
No. 4 (Mcrostie,1965)	59.80	2.43	1.70	57.36
No. 5 (Mcrostie,1965)	58.30	0.79	1.52	57.51
No. 6 (Mcrostie,1965)	58.79	0.60	2.92	58.18
No. 7 (Mcrostie,1965)	58.82	2.80	2.65	56.02

Rock Quality Designation (RQD) values measured in the boreholes ranges from 46 to 99%, indicating a poor to excellent quality rock.

Photographs of the recovered bedrock cores and results of the UCS testing are presented in Appendix E.

4.6 Groundwater Conditions

Monitoring wells were sealed into borehole BH23-05, BH23-07, BH23-07A, BH26-01 and BH26-02 during the current investigation. The following table summarizes the available measured groundwater levels. Single-well response tests were carried out at BH26-01 and BH26-02 to estimate the hydraulic conductivity of the materials screened at the monitoring wells.

Table 1: Summary of Groundwater Level

Borehole Number	Geological Unit of Screened Interval	Ground Surface Elevation (m)	Groundwater Level		Measurement Dates	Hydraulic Conductivity (m/s)
			Depth below ground surface (m)	Elevation (m)		
BH26-01	Limestone Bedrock	61.26	1.63	59.63	May 01, 2026	4×10^{-6}
BH26-02	Limestone Bedrock	61.04	1.19	59.85	May 08, 2026	2×10^{-6}
BH23-05 (shallow)	Silty Sand	62.13	(dry to 1.98)	(dry to 60.15)	September 5, 2023	-
BH23-05 (Deep)	Limestone Bedrock	62.13	3.51	58.62	September 5, 2023	-
			1.45	60.68	May 01, 2026	-
BH23-07	Silty Sand	63.38	(dry to 2.44)	(dry to 60.94)	September 5, 2023	-
BH23-07A	Limestone Bedrock	63.38	3.59	59.79	October 12, 2023	-

Notes: (1) Boreholes BH23-07 and BH23-07A were buried under a shipping container and were not accessible during the second round of measurements on May 8, 2026. Borehole BH23-05 (shallow) was not located and was likely buried under mud.

Groundwater levels are expected to fluctuate seasonally and over shorter periods of time. Higher groundwater levels are expected during wet periods of the year, such as spring after the snowmelt or during periods of heavy rain. The hydraulic conductivity of the limestone bedrock was estimated to range from 2×10^{-6} to 4×10^{-6} m/s.

5.0 DESIGN AND CONSTRUCTION CONSIDERATIONS

5.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the proposed expansion based on our interpretation of the borehole information and project requirements. Reference should be made to the “Important Information and Limitations of This Report”, which follows the text but forms part of this document.

The information in this portion of the report is provided for planning and design purposes for the guidance of the design engineers and architects. The recommendations provided herein are consistent with the Ontario Building Code of 2024 (OBC 2024). Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for

construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, schedule, safety, and equipment capabilities, costs, sequencing and the like.

5.2 Site Grading

It is understood that the proposed development will maintain approximately the existing grades, as it will be tied into the existing site development. Based on the conditions encountered, there would not be any meaningful restrictions on localized minor grade changes which may be required.

5.3 Frost Susceptibility

The fill and glacial till encountered at this site are considered to be frost susceptible. Foundations should be provided with a minimum of 1.8 m of earth for frost protection purposes. The bedrock at the site is not typically considered to be frost susceptible (provided it does not contain extensive seams of soil, which can be present in the upper portion of the bedrock).

Structures which are founded on bedrock at a depth of less than 1.8 m could be designed with reduced soil cover, however, the quality of the bedrock at the specific location would need to be confirmed during construction. This is typically done by drilling small diameter (50 mm) holes below the proposed foundation subgrade to observe the rock quality within the frost depth.

As an alternative to frost cover, insulation may also be considered. Appropriate insulation details would need to be developed during detailed design based on the specific structure, depth, location, etc.

5.4 Seismic Design

5.4.1 Seismic Considerations

Multichannel Analysis of Surface Waves (MASW) tests were carried out at the site to evaluate the average wave velocity of the upper 30 m (V_{s30}) of soil/bedrock profile. The shear wave velocities measured at the site are presented in a technical memorandum (see results in Appendix G). The results indicate that the average shear wave velocity in the upper 30 m of the subsurface at the MASW location ranges between approximately 884 and 1008 m/s, and according to 2020 NBCC site classification for seismic site response that this site can be assigned a Site Class B.

5.4.2 Seismic Hazard Values

Seismic hazard values for the proposed project site were obtained from Natural Resources Canada (2020 National Building Code of Canada Seismic Hazard Tool – <https://www.seismescanada.mcan.gc.ca/hazard-alea/interpolat/nbc2020-cnbc2020-en.php>¹). The seismic hazard values for 2% probability of exceedance in 50 years are presented below:

Table 2: Summary of Seismic Hazard Values for 2% Probability in 50 years at Cleary Development Project

Site Designation V_{s30} (X_v)	$S_a(0.2)$ (g)	$S_a(0.5)$ (g)	$S_a(1.0)$ (g)	$S_a(2.0)$ (g)	$S_a(5.0)$ (g)	$S_a(10.0)$ (g)	PGA (g)	PGV (m/s)
X_{884}	0.494	0.237	0.117	0.0531	0.0141	0.00505	0.279	0.165
X_{1008}	0.449	0.226	0.113	0.0516	0.0138	0.00501	0.284	0.160

¹ 2020 National Building Code of Canada Seismic Hazard Tool assessed on May 12, 2026

5.5 Foundation Design

As discussed previously, the subsurface conditions at this site consist of fill overlying by glacial till. It is understood that the proposed buildings will have two underground parking levels.

The bedrock surface was found to be approximately 1 to 2 m below the existing ground surface (i.e., elevations ranging from 59.9 to 62.1 m). With two levels of underground parking, the new building foundations are expected to be founded within limestone bedrock. Horizons of shale interbeds were observed within the founding depth, between approximately 6.4 and 7 m below ground surface.

It is expected the towers could be supported on pad, strip or raft foundations placed on the bedrock at the base of the basement excavation. If a shale interface is encountered following excavation at founding depth, it is recommended that a lean concrete leveling course be placed to provide a uniform bearing surface.

The lean concrete should typically have a minimum thickness of 75 to 100 mm and a minimum compressive strength of 10 MPa. The surface should be properly prepared and cleaned prior to placement to ensure adequate bonding and to reduce the potential for loosening or deterioration of the shale surface due to exposure.

Foundations supported directly on the bedrock may be designed using a factored Ultimate Limit States bearing resistance of 6 MPa. Provided the bedrock surface is properly cleaned of soil and loose rock at the time of construction, the settlement of footings sized using this factored bearing resistance would be expected to be less than the 25 mm which is typically accepted and therefore Serviceability Limit States (SLS) typically do not govern the design of shallow foundations on rock.

Foundations should be entirely supported on rock. If the existing rock surface is below the planned footing level at the time of construction (for example where a previous excavation was present), mass concrete should be placed to bring the surface up to the planned underside of footing. Mass concrete, if used, should extend beyond the edge of the footing a distance equal to the depth of the mass concrete. Alternatively, the foundation could be lowered to the as-found rock surface.

5.6 Earthworks

5.6.1 Excavations and Site Servicing

It is understood that the excavation for the proposed building towers will extend two levels below the existing ground surface. It is understood that the excavation for the proposed building towers will extend two levels below the existing ground surface, to approximately Elevation 54.9 m to 54.0 m. Excavations for the proposed site services are generally expected to extend to approximately Elevation 58.5 m to 58.9 m, based on the design invert elevations shown on the latest civil drawings prepared by LRL Engineering and dated September 29, 2026. Some service elevations and connection locations remain subject to coordination with the architectural and mechanical designs and/or field confirmation of existing utilities.

Based on the subsurface conditions encountered during the investigation, the upper portion of the proposed excavations within the overburden are expected to extend through fill and glacial till deposits.

No unusual problems are anticipated with excavating in the soil using conventional hydraulic excavating equipment. Cobbles and boulders should be expected in the glacial till. Fill material is, by nature, heterogeneous and obstructions may be encountered within the fill material.

In general, it is anticipated that open-cut methods will be feasible for excavations in soil. Where space is restricted, steel trench boxes, possibly in conjunction with steel plates and/or unsupported slopes at the surface, or a shoring system may be required to accommodate excavations.

The soils were found to be above the groundwater table and would generally be classified as a Type 3 soil in accordance with the Occupational Health and Safety Act (OHSA) of Ontario. As such, these excavations may be made with side slopes at 1 horizontal to 1 vertical (1H:1V).

5.6.2 Bedrock Excavation

Localized or shallow bedrock excavation can typically be completed by mechanical methods (such as hoe ramming). More extensive bedrock removal is more economically carried out by controlled blasting. Both methods should employ line drilling around the perimeter to define the extent of excavation and prevent overbreak.

If blasting is considered, blast induced damage to the bedrock must be avoided in the vicinity of existing structures (including buried structures such as the utilities), otherwise additional rock reinforcement could be required. At the final rock line, the bedrock should be line drilled at a close spacing in advance of blasting so that a clean bedrock face can be formed. It is considered that 75 mm diameter holes at a spacing of 200 mm or less would be appropriate for this purpose, though this is dependant on the blasting program and will need to be confirmed as part of the overall blast design.

Based on the quality of the bedrock encountered in the boreholes, it is expected that existing near vertical bedrock walls can likely be maintained for the construction period provided that any loose pieces of the bedrock are scaled off the faces for worker safety. Where the localized new excavations extend deeper than 1.5 m into the bedrock, the near vertical walls should be reviewed by a geotechnical engineer for any sign of unstable pillars or slabs that should be removed or stabilized. Stabilization options could consist of rock anchors, mesh, shotcrete, sloping the side slopes or a combination thereof. The appropriate stabilization methodology, if required, will depend on the actual site conditions during construction, and further guidance can be provided at that time.

5.6.3 Vibration Monitoring

Due to the close proximity of the existing surrounding structures to the proposed development, construction vibration, (particularly when blasting, breaking rock, driving piles or carrying out other similar vibration intensive works) should be controlled to limit the peak particle velocities at all adjacent structures or services such that vibration induced damage will be avoided.

A pre-construction survey is recommended to be carried out on all nearby structures and services. Any area of concerns should be identified during the pre-construction survey and should be monitored for movements during construction.

If blasting is required, the contractor should be required to submit a complete and detailed blasting design, as well as a monitoring plan prepared by a blasting/vibration specialist before starting blasting. This should be reviewed and accepted in relation to the requirements of the blasting specifications. The contractor should be limited to only small, controlled blasts. Peak vibration limits dependent on the following frequencies to the nearest structures and services are suggested.

The following frequency dependent peak vibration limits at the nearest structures and services are typical, but it is suggested they be confirmed by the structural engineer for the particular structure.

Frequency Range (Hz)	Vibration Limits (mm/s)
< 10	5
10 to 40	5 to 50 (sliding scale)
> 40	50

These limits should be practical and achievable on this project. Blasting typically generates vibrations greater than 40 Hz at near distances.

These limits are based on reducing the risk of structural damage to normal structures, in normal condition. These vibration limits may need to be adjusted if, for example, there is vibration-sensitive equipment in any of the receptors, the nearby structures are of unusual construction or are fragile or in poor condition (for example older, heritage structures).

It is recommended that the monitoring of ground vibration intensities (peak ground vibrations and accelerations) from the construction activities (e.g., blasting) be carried out both in the ground adjacent to the closest structures and within or at the structures themselves. Where practical, blasting should be commenced furthest from most critical receptors to allow monitoring of the ground response and (if required) adjustment of the blasting program.

5.7 Groundwater Control

It is understood that two underground parking levels are being considered, which will be located within the footprint of buildings. The parking garage at both buildings will extend to approximately 6.4 m to 7.0 m below the existing ground surface (i.e., base elevation of 54.9 m to 54.0 m). Accordingly, excavation to this depth will be through surficial fill and sand, into the underlying bedrock in areas outside the footprint of the proposed buildings footprint.

Groundwater was encountered within the boreholes advanced at this site at about 1.2 m below ground surface in May 2026. Drainage is expected below the floor slab level at 6.4 m and 7.0 m at each building. During construction, excavation will extend to the bedrock (elevation of 54.9 m to 54.0 m) and dewatering will be required to that level. Additional excavations will also be required for the installation of the services for the structure. Based on the groundwater conditions observed on site, excavations will extend below the groundwater level and, as such, groundwater control will be required.

According to O.Reg. 63/16 and O.Reg. 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 litres per day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a Water Taking Plan and a Discharge Plan.

For preliminary dewatering assessment purposes, the construction of the proposed 2-level underground parking garage is assumed to be completed using an open excavation. To estimate the groundwater taking rate required to complete the proposed works, the equation for groundwater inflow to a circular excavation in an unconfined aquifer (Dupuis-Forchheimer Equation) was used.

For purposes of construction dewatering estimates the following assumptions were made:

- Unconfined aquifer conditions.
- Line source and radial groundwater flow under steady state conditions.
- The excavation is assumed to be circular, and an equivalent radius is calculated based on the actual area of the proposed excavation. The total area of the proposed excavation required to construct the 2-level parking garage is assumed to be approximately 3,400 m², which corresponds to an equivalent radius of approximately 33 m. The depth of the excavation was assumed to extend to an elevation of 54 masl (refer to Appendix F for attached dewatering calculations).
- The initial groundwater elevation within the excavation area was assumed to be 60.68 masl (i.e., the highest groundwater elevation measured between at BH23-05 (Deep) in May 2026 as shown in Table 1).

- Groundwater will flow into the excavation through existing fill, native overburden and fractures in weathered limestone bedrock. The highest hydraulic conductivity measured within the bedrock at the site (4×10^{-6} m/s, as estimated at BH26-01) was used to provide a conservative (i.e., high-end) estimate of groundwater inflow.

The assumptions provide a preliminary estimate for the groundwater inflow into the proposed excavation for the construction of the proposed 2-level underground parking garage. Based on the assumptions provided above, the results of steady-state flow rate of groundwater inflow into a single circular excavation with an equivalent radius of 33 m, to be used for water taking registrations (i.e. EASR), is estimated to be about 79,000 L/day. Steady-state flow rates represent water taking rates required to maintain dry conditions within the excavation area once the excavation area is fully dewatered. Initial rates of groundwater inflow to the excavation are expected to exceed the steady-state rate, and are estimated to be about 1,060,000 L/day. The estimated radius of influence (ROI) associated to the dewatering of the open excavation using these parameters is approximately 40 metres.

Based on the soil and groundwater conditions as well as the size of the proposed excavation, the construction of the 2-level parking garage will likely require an EASR (along with a Water Taking Plan and a Discharge Plan) as the estimated steady-state flow rate of groundwater into the excavation is anticipated to be greater than 50,000 L/day. This assumption should be reviewed during detailed design based on the actual proposed excavations.

5.8 Basement Drainage

The measured groundwater depth at the site is variable, but it is generally considered to be between about 1.2 m (Elev. 59.9) to 3.4 metres (Elev. 58.8) below existing site grades within the proposed buildings locations.

The backfill and drainage requirements for basement walls, as well as the lateral earth pressures will depend on the exact details of the existing excavation and the new underground structure.

Drainage, such as a composite synthetic drainage system or equivalent, should be provided to the exterior walls above elevation 54.9 m and 54.0 m, for each proposed building, respectively. The composite drain must withstand the design horizontal earth pressures used for basement wall design and should be connected to the foundation drains at that level. The drainage system collector pipes should drain to a sump for collection and discharge to a sewer.

Permanent structures that extend below the drainage level (i.e., 54.9 m or 54.0 m elevation) should be designed and constructed to be watertight and will need to resist the uplift forces generated from buoyancy.

5.8.1 Open Cut Excavations

The soils at this site are potentially frost susceptible and should not be used as foundation backfill. To avoid problems with frost adhesion and heaving as well as to provide drainage, these foundation elements should be backfilled, within the design frost penetration depth below finished grade, with non-frost susceptible sand or sand and gravel conforming to the requirements for Ontario Provincial Standard Specification (OPSS) Granular B Type I, Granular B Type II, or Granular A.

To avoid ground settlements around the basement walls which could affect site grading and drainage, all of the backfill materials should be placed in 0.3 m thick lifts and compacted to at least 95% of the material's SPMDD.

The basement wall backfill should be drained by means of a perforated pipe subdrain in a surround of 19 mm clear stone, fully wrapped in a geotextile, which leads by positive drainage to a storm sewer or to a sump from which the water is pumped.

5.9 Basement Floor /Raft Slab

The basement floors are expected to be located below the groundwater level measured at the site, which fluctuate between approximately 1.2 m and 3.4 m below existing grade. In addition, it is likely that, over the life of the building, groundwater levels may exceed those recorded during the field investigation.

As a precautionary measure, it is recommended that subdrainage be installed beneath the basement slab. The subdrainage system may consist of perforated pipe drains, as discussed above. For preliminary design, the drains may be assumed to be spaced at approximately 6 m intervals.

5.10 Lateral Earth Pressures for Design

The magnitude of the lateral earth pressures will depend on the backfill materials and backfill conditions adjacent to the foundation walls. If the backfill materials for open cut excavations consist of compacted granular and the walls are considered rigid (i.e., no lateral displacement), then the lateral earth pressures may be taken as:

$$s_h(z) = K_o (gz + q)$$

Where:

- $s_h(z)$ = Lateral earth pressure on the wall at depth z , kPa;
- K_o = At-rest earth pressure coefficient for rigid walls, see table below;
- K_a = Active earth pressure coefficient, see table below;
- g = Unit weight of retained soil, use 21 kN/m³;
- z = Depth below top of wall, metres; and,
- q = Uniform surcharge at ground surface to account for traffic and equipment (not less than 15 kPa), plus any surcharge due to adjacent foundation loads.

The following values in the table below provide preliminary guidelines for the lateral earth pressures for static (i.e., not earthquake) loading conditions for planning purposes. These lateral earth pressure coefficients assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes, new lateral earth pressures will need to be calculated (or the soil above the wall treated as a surcharge).

Table 3: Lateral Earth Pressure Coefficient for Unrestrained Shoring

Strata	Soil Unit Weight ρ (kN/m ³)	Effective Angle of Friction, ϕ' (°)	Coefficients of static lateral earth pressure		
			Active coefficient, K_a	Passive Coefficient, K_p	At rest, K_o
Existing Granular or Native Sand	23	32	0.31	3.25	0.47
Granular A	21	35	0.27	3.69	0.43
Granular B Type II	22	35	0.27	3.69	0.43

These lateral earth pressures would increase under seismic loading conditions. The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) for design may be determined as follows:

$$s_h(z) = K_o \gamma z + (K_{AE} - K_a) \gamma (H - z)$$

- Where:
- $s_h(z)$ = Lateral earth pressure at depth z , kilopascals;
 - K_{AE} = Seismic earth pressure coefficient, see table below, and;
 - H = Total height of the wall, metres.

Table 4: Seismic Active Coefficients (K_{AE})

Material	Seismic Active Pressure Coefficients, K _{AE}		
	Existing Granular Fill or Native Sand	Granular A	Granular B Type II
Yielding wall	0.40	0.35	0.35
Non-Yielding wall	0.51	0.46	0.46

It should be noted that all of the lateral earth pressure equations are given in an unfactored format and will need to be factored for ULS design purposes.

If the wall allows lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

- Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
- Horizontal translation of 0.001 times the height of the wall; or,
- A combination of both.

If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

5.11 Rock Anchors

If required, rock anchors could be provided to resist uplift loads on the foundations and slabs (such as due to seismic forces on the building or hydrostatic pressures), or to increase the sliding resistance. The anchors could consist of either grouted or mechanical anchors.

The design of the rock anchors should be generally carried out in accordance with the guidelines provided in the CFEM and FHWA-IF-99-015 (Section 5.9.2 and other sections). The design and construction of tiedown rock anchors should also follow the specifications in OPSS.MUNI 942.

Rock anchors are typically installed in a borehole that is drilled with air-percussion equipment or with rotary diamond drilling equipment with water circulation; those drilling methods can fairly penetrate through boulder/cobbler ground as well as bedrock. A cased hole would be drilled through the overburden with a socket drilled into the bedrock, the steel anchor inserted, and then the annular space around the bar filled with grout.

Because the rock anchors would be permanent elements of the foundations, a “double corrosion protection” system should be provided.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes.

- i) failure of the steel tendon or top anchorage
- ii) failure of the grout/tendon bond
- iii) failure of the rock/grout bond
- iv) failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by the structural engineer. Adequate corrosion protection of the steel components should be provided to prevent potential premature failure due to steel corrosion, particularly in the submerged environment at the site.

For potential failure mode iii), the unfactored ULS bond stress at the concrete/rock interface may be taken as **1,500 kPA**. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, for a preliminary assessment it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance should be calculated based on the buoyant weight of the potential mass of rock which could be mobilised by the anchor. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \phi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

where:

- Q_r = factored uplift resistance of the anchor, kilonewtons
- ϕ = resistance factor, 0.3
- γ' = effective unit weight of rock, use 26 kN/m³ above groundwater level, 16 kN/m³ below groundwater level
- D = anchor length in metres
- θ = Half of the apex angle of the rock failure cone, use 30 degrees

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r' = Q_r \cos(\alpha)$$

Where:

- Q_r' = Factored uplift resistance of the anchor subject to inclined load (kN)
- Q_r = Factored uplift resistance of the anchor (kN)
- α = Angle between the load direction and the vertical

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 \sin^2 \theta + a D^2 \sin \theta + b D^2 \sin \theta + a b D$$

- Where: V = Volume of the truncated trapezoid failure zone (m³)
- D = Depth of anchor group (m)
- a = Width of anchor group (m)
- b = Length of the anchor group (m)
- θ = One-half of the apex angle of the rock failure cone (use 30°)

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \phi \gamma' V$$

Where: Q_r = Factored uplift resistance of the anchor (kN)
 ϕ = Resistance factor, use 0.4
 γ' = Effective unit weight of rock (use 16 kN/m³)
 V = Volume of truncated trapezoid (m³)

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained. This assessment, however, requires a detailed understanding of the anchor lengths, geometry, loads, etc. and would need to be completed during detailed design.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out in accordance with OPSS 942 (*Prestressed Soil and Rock Anchors*).

A geotechnical professional should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids. Confirmation of sufficient embedment into the rock beneath the foundations should be carried out to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

5.12 Pavement Design

It is understood new parking lots and access roadway will be constructed as part of the development. The following provides guidelines for the construction of access roads and parking areas for light traffic (i.e. cars) and heavy traffic (i.e. trucks).

5.12.1 Subgrade Preparation

In preparation for pavement construction, all topsoil, unsuitable fill, disturbed, or otherwise deleterious materials (i.e., those materials containing organic material) should be removed from the pavement areas. Some of the existing fill could remain provided that it is free of organic matter, and that the subgrade be subjected to a proof roll with a loaded tandem truck to reveal weak or soft areas prior to the construction of the new pavement structure. Soft or weak areas should be removed and repaired with acceptable earth borrow or OPSS Select Subgrade Material (SSM).

Sections requiring grade raising to the proposed subgrade level should be filled using acceptable (compactable and inorganic) earth borrow (OPSS.MUNI 206/212), Select Subgrade Material (OPSS.MUNI 1010) or additional granular base if grade changes are minor. These materials should be placed in maximum 300 mm thick lifts and should be compacted to at least 98% of the materials SPMDD using suitable compaction equipment.

The surface of the subgrade or fill should be crowned or sloped to promote drainage of the roadway granular structure. Perforated pipe subdrains should be provided along the low sides of the roadway along the entire length. The subdrains should be installed in accordance with OPSS.MUNI 405. The subdrains should be connected to the catch basins such that the pavement structure will be positively drained and will intercept flows within the subbase.

Below the pavement structure, frost compatibility must be maintained across any new service trenches. Due to the variability of the soils within the project limits, the subsoil should be inspected by qualified geotechnical personnel to make sure that there is no potential for differential frost heaving. Frost tapers from the bottom of granular subbase to 1.8 m depth should be constructed at 10H:1V and should be provided where necessary.

The pavement recommendations have been split up into two categories of light duty and heavy-duty pavements. It has been assumed the light duty areas will consist of parking areas and lighter vehicles (i.e., no truck or bus traffic), and the heavy-duty pavements will consist of occasional truck traffic. The pavement in each area should be constructed as follows:

Material		Thickness of Pavement Elements (mm)	
		Light Duty	Heavy Duty
Asphaltic Concrete OPSS.MUNI 1151	Superpave 12.5 mm	40	50
	Superpave 19.0 mm	50	70
Granular Material OPSS.MUNI 1010	Granular A Base	150	150
	Granular B, Type II Subbase	400	500

The above pavement design is based on the assumption that the pavement subgrade has been acceptably prepared (i.e., where the bottom of the excavation has been adequately compacted to the required density and the subgrade surface is not disturbed by construction operations or precipitation). Depending on the actual conditions of the pavement subgrade at the time of construction, it could be necessary to increase the thickness of the subbase.

Additionally, a Class II woven geotextile conforming to OPSS 1860 should be provided under pavement areas to prevent pumping of the subgrade into the Granular B Type II subbase.

5.13 Corrosion and Cement Type

Two soil samples from boreholes 23-03 and 23-05 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results indicate that concrete made with Type GU Portland cement should be acceptable for substructures.

The results of the current investigation also indicate a low potential for corrosion of buried ferrous elements, which should be considered in the design of substructures, buried ferrous elements.

The results of this testing are provided in Appendix D and summarized below.

Borehole Number	Sample Number	Depth Intervals (m)	Chlorides (%)	Sulphates (%)	pH	Resistivity (Ohm-cm)
23-03	2	0.76 – 1.37	0.005	0.01	8.07	3448
23-05	3	1.52 – 2.13	0.006	0.01	8.76	4348

6.0 TREE PLANTING

The subsurface conditions at the site generally consist of granular soils, predominantly sand and gravel, overlying bedrock. As such, the potential for significant soil shrinkage associated with moisture depletion by trees is considered limited relative to sites underlain by sensitive cohesive soils. Where sensitive, cohesive soil deposits are encountered, consideration must be given to potential moisture-related volume changes when selecting tree

species and establishing planting locations. Tree planting should also be coordinated with the proposed foundations and underground structures.

Based on the predominant subsurface conditions, sensitive clay deposits are not present at the site and no specific geotechnical restrictions on tree planting adjacent to the proposed buildings or the existing daycare building are anticipated. However, tree locations should be coordinated with building foundations to minimize the potential for interference from root systems. Tree planting above the underground parking garage should also be coordinated with the structural and landscape design, considering the available soil depth, anticipated tree loads, drainage requirements, and protection of the underground structure and waterproofing system.

7.0 CONSTRUCTION CONSIDERATIONS & UTILITY PROTECTION

7.1 Impact on Adjacent Developments

Possible impacts on adjacent developments could result from:

- Ground movement around the perimeter of new excavations.
- Ground settlements due to the planned temporary and permanent groundwater level lowering.

A preconstruction survey of all structures located within close proximity to this site should be carried out prior to commencement of the excavation.

Given the relatively shallow depth of additional bedrock excavation, underpinning or extensive shoring is not expected to be required for the proposed construction. The proposed excavation extents should, however be reviewed during detailed design to confirm this assumption. During construction the exposed bedrock should be inspected by qualified geotechnical personnel at the time of excavation, particularly in areas where excavations will be in close proximity to existing foundations.

7.2 Impact on Adjacent Municipal Water Infrastructure

A critical, large-diameter concrete backbone watermain is located immediately adjacent to the boundary of the subject site. Geotechnical hazards associated with construction adjacent to this type of rigid utility include ground-borne vibration transmission, lateral strain development from stress relief during excavation, and differential settlement or joint rotation from subgrade disturbance. Left unmitigated, these factors could compromise the structural integrity of the watermain or induce localized joint failure.

7.3 Recommendation for Instrumentation and Monitoring

Based on the proximity of the proposed excavation to the municipal infrastructure, development of a formal Vibration and Settlement Monitoring Program is strongly recommended and will likely be required by the City of Ottawa as a condition of approval.

Prior to any site mobilization, a comprehensive pre-construction condition survey must be completed. During construction, automated monitoring must be conducted continuously to verify that ground movements and vibration levels remain within acceptable engineering tolerances.

7.4 City of Ottawa Drinking Water Services Minimum Requirements

To ensure the structural integrity of the adjacent municipal concrete backbone watermain, all construction activities must strictly adhere to the following City of Ottawa protective mandates:

- **Mechanical Excavation Exclusion:** Mechanical excavation is prohibited within 3.0 m of the outer face of the concrete watermain. Non-destructive methods (e.g., hand-digging, hydro-vacuuming) must be utilized within this zone in compliance with the [City of Ottawa UCC Horizontal and Vertical Clearance Guidelines Matrix](#).
- **Engineered Subgrade Support:** Any excavation executing below the watermain invert requires a site-specific support and underpinning plan designed and stamped by a Professional Engineer licensed in Ontario, incorporating continuous vertical displacement monitoring.
- **Displacement Monitoring:** Continuous horizontal movement tracking is mandatory for any shoring installation or bulk excavation occurring at or below the established watermain elevation.
- **Equipment Load Distribution:** Heavy machinery must not operate directly above the watermain alignment. Temporary, structurally rated steel plates must be deployed at all designated utility crossing points to distribute traffic loads.
- **Seismograph Spatial Array:** Continuous, high-accuracy seismographs must be installed directly on or adjacent to the utility asset, spaced at maximum 10 m intervals along the construction corridor.
- **Structural Performance Criteria:** Peak Particle Velocity (PPV) limits must comply with City of Ottawa Special Provision (S.P.) F-1201 as outlined in the [City of Ottawa Blasting and Construction Vibration Guidelines](#) (maximum 20 mm/s for low-frequency waves <40 Hz; 50 mm/s for high-frequency waves >40 Hz).
- **Response Framework & Tiered Thresholds:** A rigid engineering response plan must be enforced under the jurisdictional authority of the [City of Ottawa Water By-law No. 2025-227](#), based on the following limits:
 - **Warning Limit (75% of Maximum):** Triggered at 15 mm/s (low frequency) / 37.5 mm/s (high frequency). Requires an immediate modification of construction methods to lower vibration outputs.
 - **Maximum Limit (100% of Maximum):** Set at 20 mm/s / 50 mm/s. Any exceedance mandates an immediate stop-work order on all excavation and compaction activities, notification to City Engineering Services, and a formal structural assessment.

8.0 ADDITIONAL CONSIDERATIONS

At the time of writing this report, only conceptual details related to the proposed building towers were available. WSP should review the final drawings and specifications for this project prior to tendering to confirm that the guidelines in this report have been adequately interpreted.

During construction, sufficient foundation inspections, subgrade inspections, in-situ density tests, materials testing, pile installation monitoring should be carried out to confirm that the conditions exposed are consistent with those encountered in the borehole, and to monitor conformance to the pertinent project specifications. Concrete testing should be carried out in a CCIL certified laboratory.

All bearing surfaces must be inspected by WSP prior to filling or concreting to ensure that strata having adequate bearing capacity have been reached and that the bearing surfaces have been properly prepared.

9.0 CLOSURE

WSP trust that this report provides sufficient geotechnical engineering information to facilitate the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Signature Page

WSP Canada Inc.



Arthur Kuitchoua Petke, P.Eng
Geotechnical Engineer

Chris Hendry, P.Eng
Senior Geotechnical Engineer

AKP/CH/al/yj

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IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to WSP by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. WSP cannot be responsible for use of this report, or portions thereof, unless WSP is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without WSP's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, WSP may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to WSP. The report, all plans, data, drawings and other documents as well as all electronic media prepared by WSP are considered its professional work product and shall remain the copyright property of WSP, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of WSP. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of WSP's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that WSP interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: WSP will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of WSP's report. WSP should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of WSP's report.

During construction, WSP should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of WSP's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in WSP's report. Adequate field review, observation and testing during construction are necessary for WSP to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, WSP's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

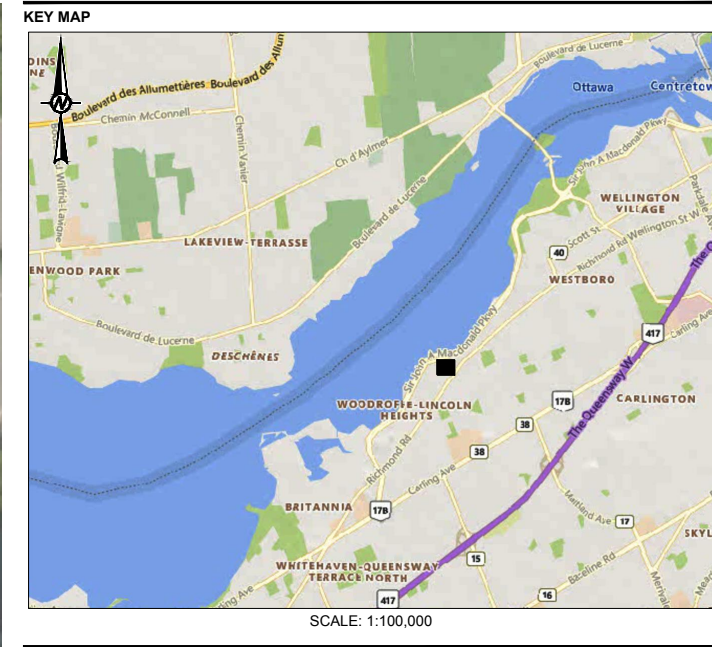
Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that WSP be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

FIGURES

Figure 1 – Site Plan

Path: \\gdrive.google.com\userdata\office\Ontario\SM\Clients\Theia_Partners\40_Proj\0001_Consult\1_File Name: CA0008376-9447_Theia_Partners\40_Proj\0001-EG-0001.dwg | Last Edited By: albaneseldcawar Date: 2023-09-21 Time: 2:17:32 PM | Printed By: SAA Date: 2023-11-03 Time: 2:02:49 PM



- LEGEND**
- APPROXIMATE BOREHOLE LOCATION, CURRENT INVESTIGATION
 - APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY MCRSTIE & ASSOCIATES LTD., REPORT NO. SF-763 (1964)
 - APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY MCRSTIE SETO GENEST & ASSOCIATES LTD., REPORT NO. SF-917 (1965)

- REFERENCE(S)**
- PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18



CLIENT			
THEIA PARTNERS			
PROJECT			
GEOTECHNICAL INVESTIGATION			
TWO MULTI-USE RESIDENTIAL BUILDINGS			
30 CLEARLY AVENUE, OTTAWA, ONTARIO			
TITLE			
SITE PLAN			
CONSULTANT		YYYY-MM-DD	2023-09-05
		DESIGNED	----
		PREPARED	NV
		REVIEWED	----
		APPROVED	----
PROJECT NO.	CONTROL	REV.	FIGURE
CA0008376.9447	0001	A	1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM A4 (210 x 297 mm) TO A3 (297 x 420 mm). 25 mm

APPENDIX A

**Record of Borehole Sheet
(Current Investigation)**

METHOD OF SOIL CLASSIFICATION

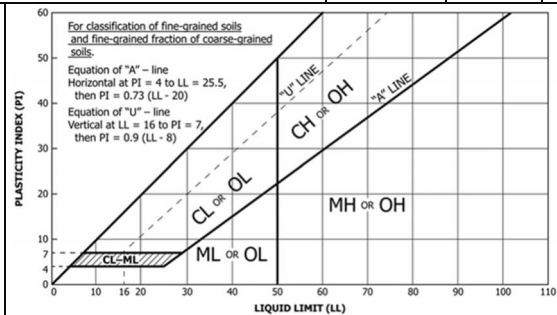
The WSP Canada Soil Classification¹ System is based on the Unified Soil Classification System (USCS) (after ASTM D2487)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$C_u = \frac{D_{60}}{D_{10}}$	$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$	Organic Content ^{6,9}	USCS Group Symbol ^{3,5,7}	Primary Group Name ²
INORGANIC (Organic Content <30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Clean Gravels with <5% fines ³ (by mass)	Well Graded	≥4 (and)	≥1 to ≤3	≤30%	GW	Well-graded GRAVEL ^{4,6}
				Poorly Graded	<4 (and/or)	<1 or >3		GP	Poorly graded GRAVEL ^{4,6}
			Gravels with >12% fines ³ (by mass)	Below A Line	n/a			GM	SILTY GRAVEL ^{4,6}
				Above A Line	n/a			GC	CLAYEY GRAVEL ^{4,5,6}
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Clean Sands with <5% fines ⁷ (by mass)	Well Graded	≥6 (and)	≥1 to ≤3		SW	Well-graded SAND ^{6,8}
				Poorly Graded	<6 (and/or)	<1 or >3		SP	Poorly graded SAND ^{6,8}
			Sands with >12% fines ⁷ (by mass)	Below A Line	n/a			SM	SILTY SAND ^{6,8}
				Above A Line	n/a			SC	CLAYEY SAND ^{5,6,8}

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators					Organic Content ^{8,H}	USCS Group Symbol ^A	Primary Group Name ^A
				Dilatancy	Dry Strength	Shine Test	Thread Diameter (mm)	Toughness (of 3 mm thread)			
INORGANIC (Organic Content <30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Nonplastic or PI and LL plot below A-Line on Plasticity Chart below) ^C	Liquid Limit <50 ^D	Rapid	None to Low	Dull to None	3 to >6	Low/can't roll 3 mm	<15%	ML	SILT ^H
				None to Slow	Low to Medium	Dull to Slight	3 to 6	Low	15% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50 ^D	None to V.Slow	Low to Medium	Slight	3 to 6	Low to Medium	<15%	MH	ELASTIC SILT ^H
				None	Medium to High	Dull to Slight	1 to 3	Low to Medium	15% to <30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below) ^A	Liquid Limit <50 ^D	None to Medium Slow	Medium to High	Slight to Shiny	1 to 3	Medium	<15%	CL	LEAN CLAY ^{A,E,F,G,H}
				None to V.Slow	Medium to High	Slight to Shiny	1 to 3	Medium	15% to <30%	OL	ORGANIC CLAY ^{E,F,G}
			Liquid Limit ≥50 ^D	None	High to V.High	Shiny	<1	High	<15%	CH	FAT CLAY ^{E,F,G,H}
				None	High	Shiny	<1 to 1	High	15% to <30%	OH	ORGANIC CLAY ^{E,F,G}
HIGHLY ORGANIC SOILS (Organic Content ≥30% by mass)		Peat and mineral soil mixtures	Relatively lightweight, possibly spongy. Some water may squeeze from sample. Some shrinkage may occur on air drying. Sand fraction may be visible. Low to high dilatancy. Thread weak near plastic limit. Low to medium dry strength.						30% to <75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat	Lightweight, spongy. Much water squeezes from sample. Shrinks considerably on air drying (i.e., very high water content). Plant structure identifiable to altered.						75% to 100%		PEAT

Coarse-Grained Soil Note(s):

- Based on the material passing the 75 mm sieve.
- If field sample contains or drilling observations indicate cobbles or boulders or both, add, "with cobbles" or "with cobbles and boulders". Include notes on the depth(s) encountered, and sizes if possible.
- Gravels with 5% to 12% fines require dual symbols:
(GW-GM) Well-graded GRAVEL with silt,
(GW-GC) Well-graded GRAVEL with clay,
(GP-GM) Poorly graded GRAVEL with silt,
(GP-GC) Poorly graded GRAVEL with clay.
- If soil contains ≥15% sand, add "with sand" to Group Name.
- If fines classify as CL-ML, use dual symbol (GC-GM) or (SC-SM) for Group Symbol.
- If the soil has an organic content (OC) 15%≤OC<30% the prefix "Organic" should be added before the Group Name. If the soil has an organic content 3%≤OC<15% add "with organic fines" to Group Name. If the soil contains >0% to ≤3% organics, the descriptor "trace organics" may be added to the Group Name.
- Sands with 5% to 12% fines require dual symbols:
(SW-SM) Well-graded SAND with silt,
(SW-SC) Well-graded SAND with clay,
(SP-SM) Poorly graded SAND with silt,
(SP-SC) Poorly graded SAND with clay.
- If soil contains ≥15% gravel, add "with gravel" to Group Name.



Fine-Grained Soil Note(s):

- If Atterberg limits plot above the A-line but in the 'hatched' area on the plasticity chart, soil is a (CL-ML) SILTY CLAY.
- If the soil contains >0% to ≤3% organics, the descriptor "trace organics" may be added to the Group Name.
- If fine-grained materials are nonplastic (i.e., a plastic limit (PL) cannot be measured), soil is a (ML) SILT.
- If soil has a liquid limit (LL) >30% to <50%, the term 'medium plasticity' may be included in the description, but the Group Name/Symbol is not changed.
- If soil contains 15% to <30% +No.200, add "with sand" or "with gravel".
- If soil contains ≥30% +No.200 mainly sand, add "Sandy" to Group Name.
- If soil contains ≥30% +No.200 mainly gravel, add "Gravelly" to Group Name.
- If the soil has an organic content (OC) 3%≤OC<15% add "with organic fines" to Group Name.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse	19 to 75	0.75 to 3
	Fine	4.75 to 19	(4) to 0.75
SAND	Coarse	2.00 to 4.75	(10) to (4)
	Medium	0.425 to 2.00	(40) to (10)
	Fine	0.075 to 0.425	(200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

GRADATIONAL COMPONENT TERMS

% (by mass)	Term
< 5	Use "trace"
≥ 5 to ≤ 12	Use "few"
> 12 to <30	Use "little"
≥ 30 to <50	Use "some"
≥ 50	Use "mostly"

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d :

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

ROCK

RDQ (Rock Quality Designation)

Quality Designation	RQD (%)
Very poor quality	< 25
Poor quality	25 – 50
Fair quality	50 – 75
Good quality	75 – 90
Excellent quality	> 90

USUAL SAMPLES

AS	Auger sample
DD	Diamond Drilling
GS	Grab Sample
RC	Rock core
SC	Soil core
SS	Split-spoon sampler (50 mm OD); larger sizes use MC
TP	Thin-walled, piston – note size (Shelby tube)
TT	Seamless open ended, driven, pushed tube sampler, or geoprobe macro-core – note size (transparent tubing)

OTHER SAMPLES

BS	Block sample
DS	Denison type sample
EX-F	Floor sample
EX-P	Wall sample
MC	Modified California Samples – note sample diameter and hammer weight
MS	Modified Shelby (for frozen soil)
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
WS	Wash sample

STATE

Disturbed		Lost		Undisturbed		Cored	
-----------	---	------	---	-------------	---	-------	---

CALIBER

Calibre (SS)	Interior diameter (mm)	Caliber (core barrel)	Interior diameter (mm)
P	75.9	PQ	85.0
H	63.2	HQ	63.5
N	50.5	NQ	47.6
B	37.8	BQ	36.4

WELL INSTALLATION

GROUNDWATER LEVEL AND DATE OF SURVEY



LEVEL OF LIQUID IN NON-AQUEOUS PHASE AND DATE OF SURVEY



MAT.	Material for well construction
DIA.	Well diameter
OP.	Opening of the slotted screen
PVC	Polyvinyl chlorid
SCH	Schedule (width of the PVC wall)
SS	Stainless steel

OBSERVATIONS¹

Odours		Visual	
L	Low	D	Disseminated
M	Medium	S	Saturated
P	Strong	-	-

¹ The odours reported were perceived incidentally during fieldwork. No sample has been deliberately smelled.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

NON-COHESIVE (COHESIONLESS) SOILS

Compactness²

Term	SPT 'N' (blows/0.3m) ¹
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in general accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

SOIL TESTS

GSA	grain size analysis
w_N	water content
PL, w_p	plastic limit
LL, w_L	liquid limit
PI	plastic index
LI	liquidity index
C	consolidation (oedometer) test
σ'_p	consolidation pressure
C_c	compression index
C_{cr}	recompression index
e_0	initial void ratio
k	hydraulic conductivity
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
C_u	Undrained shear strength on undisturbed soil using a fall cone penetrometer
C_r	Undrained shear strength on remolded soil using a fall cone penetrometer
S_u	Undrained shear strength on undisturbed soil - field
S_r	Undrained shear strength on remoulded soil - field
R	Refusal
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
GS	specific gravity
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in general accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
$w_N < PL$	Material is estimated to be drier than the Plastic Limit.
$w_N \sim PL$	Material is estimated to be close to the Plastic Limit.
$w_N > PL$	Material is estimated to be wetter than the Plastic Limit.

Plasticity

Term	Liquid Limit (LL)
Low	< 30%
Medium	30 – 50%
High	> 50%

Sensitivity (CFEM, 2023)

Term	Sensitivity, S_r
Low	< 8
Medium	8 – 30
High	> 30

ROCK TESTS

UCS (σ_c)	uniaxial compressive strength
γ	Unit weight
ν	Poisson's ratio
E	Linear deformation modulus
E_D	Dilatometer modulus
E_M	Pressuremeter modulus

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows.

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES**(a) Index Properties**

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation
w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
NP	nonplastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_d	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes

- * Density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)
- 1. $\tau = c' + \sigma' \tan \phi'$
- 2. shear strength $= (\text{compressive strength})/2$

PROJECT: CA0008376.9447

LOCATION: N 5025707.68; E 439473.01

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-01

BORING DATE: August 24, 2023

DRILL RIG: CME 75

SHEET 1 OF 2

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³						
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT						
								100	200	300	400							
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		59.91														GR SA SI CL
		ASPHALT		59.89														
		FILL - (SP/GP) SAND and GRAVEL, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, loose			1	SS	9	ND										
1		Borehole continued on Record Drillhole BH23-01		59.05	2	SS	50/ 0.05											
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

DATUM: Geodetic

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing

[illegible]

PROJECT: CA0008376.9447

LOCATION: N 5025681.49; E 439478.54

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-02

BORING DATE: August 25, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected 100 200 300 400				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³					
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected 100 200 300 400				WATER CONTENT PERCENT Wp ——— W ——— WI 20 40 60 80					
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		60.22											GR SA SI CL		
		ASPHALT		59.89	1	SS	24	ND									
		FILL - (SP) gravelly SAND, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, compact,		59.46													
1		(SM) SILTY SAND, trace clay, trace gravel; grey to brown (GLACIAL TILL); non-cohesive, moist to wet, loose		0.76	2	SS	8										
		END OF BOREHOLE Auger Refusal		58.75													
		Note(s): 1. Borehole dry upon completion of drilling.		1.47													
2																	
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447

LOCATION: N 5025657.83; E 439481.27

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-03

BORING DATE: August 25, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕ ND = Not Detected				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □ ND = Not Detected				WATER CONTENT PERCENT					
								100	200	300	400	Wp	W			WI	20
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		60.59											GR SA SI CL		
		ASPHALT		60.00													
		FILL - (SP/GP) SAND and GRAVEL, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, compact		59.92	1	SS	16	⊕								47 45 (8)	
		(SP) SAND, trace to some silt, fine grained; brown; moist to wet, loose		0.67	2	SS	7	⊕									
1				59.07													
		(SM/GP) SILTY SAND and GRAVEL; dark to brown, contains organic matters (GLACIAL TILL); moist, very dense		1.52	3	SS	50/ 0.25	⊕									
2				58.76													
		END OF BOREHOLE Auger Refusal		1.83													
		Note(s):															
		1. Borehole dry upon completion of drilling.															
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

PROJECT: CA0008376.9447

LOCATION: N 5025645.99; E 439508.24

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-04

BORING DATE: August 25, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected 100 200 300 400				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³					
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT					
								ND = Not Detected 100 200 300 400				Wp ----- W ----- WI 20 40 60 80					GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		61.15												GR SA SI CL	
		ASPHALT		0.00													
		FILL - (GP) sandy GRAVEL, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, dense		0.08	1	SS	32										
				60.39													
1		(SM) SILTY SAND, trace clay, trace gravel; brown with black bedding (GLACIAL TILL); moist, very dense		0.76	2	SS	14										
				59.78													
		END OF BOREHOLE Auger Refusal		1.37													
2		Note(s):															
		1. Borehole dry upon completion of drilling.															
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447
LOCATION: N 5025611.77; E 439503.11

RECORD OF BOREHOLE: BH23-05

SHEET 1 OF 2
DATUM: Geodetic
HAMMER TYPE: AUTOMATIC

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

BORING DATE: August 24, 2023
DRILL RIG: CME 75

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
								ND = Not Detected				Wp — W — WI																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
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DEPTH SCALE
1 : 50



LOGGED: OB
CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINTOTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447

RECORD OF DRILLHOLE: BH23-05

SHEET 2 OF 2

LOCATION: N 5025611.77 ;E 439503.11

DRILLING DATE: August 24, 2023

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA				WEATH- ERING INDEX		Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
							TOTAL CORE %	SOLID CORE %			TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jzon	W1	W2		W3	W4	W5			W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
2	Rotary Drill NQ Core	Cont'd from Record of Borehole BH23-05		60.20 1.93																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

PROJECT: CA0008376.9447

LOCATION: N 5025623.60; E 439532.36

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-06

BORING DATE: August 25, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected 100 200 300 400				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³						
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT						
								ND = Not Detected 100 200 300 400				Wp — W — WI						
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		61.98												GR SA SI CL		
		ASPHALT		0.00														
		FILL - (GP) sandy GRAVEL, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, dense		0.13	1	SS	65	ND										
1		(SM/GP) SILTY SAND and GRAVEL; grey to brown, cobbles and boulders, contains strong petroleum odor (GLACIAL TILL); moist, compact		61.17	2	SS	23									38 41 (21)		
				0.81														
				60.48	3	SS	50/ 0.20											
2		- rock fragments END OF BOREHOLE Auger Refusal		1.50														
		Note(s): 1. Borehole dry upon completion of drilling.																
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447

LOCATION: N 5025623.43; E 439573.43

RECORD OF BOREHOLE: BH23-07

SHEET 1 OF 1

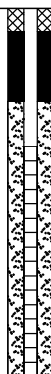
BORING DATE: August 25, 2023

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

DRILL RIG: CME 75

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES		HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION				
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT							
								ND = Not Detected				Wp — W — WI							
								100	200	300	400	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	10 ⁻³				
								100	200	300	400	20	40	60	80				
0		GROUND SURFACE		63.38												GR SA SI CL			
	Power Auger 204 mm Diam. (Hollow Stem)	FILL - (SP/GP) SAND and GRAVEL, trace silt; brown to grey, angular; non-cohesive, moist, loose (SM) - SILTY SAND, some clay, some gravel; grey to black, cobbles and boulders, contains organic matter (GLACIAL TILL); moist, compact - fine grained SAND; moist to wet																	
				0.00	1A			16	⊕	ND									
				0.15	1B	SS				⊕	ND				○				
1				2	SS			25	□	ND	⊕								
				3	SS			7	□	ND	⊕				○				
2	4	SS			50/ 0.23	□	ND	⊕				○							
				60.94															
		END OF BOREHOLE Auger Refusal		2.44															
3		Note(s): 1. Borehole dry upon completion of drilling.																	
4																			
5																			
6																			
7																			
8																			
9																			
10																			

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447

LOCATION: N 5025621.99; E 439571.86

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-07A

BORING DATE: September 29, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES		DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION				
		DESCRIPTION	STRATA PLOT	ELEV.	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH Cu, kPa		nat V. + Q - rem V. ⊕ U - ⊙		WATER CONTENT PERCENT				GRAIN SIZE DISTRIBUTION (%)				
				DEPTH (m)				20	40	60	80	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴		10 ⁻³	GR	SA	SI	CL
0		GROUND SURFACE		63.38											GR	SA	SI	CL		
		FILL - (SP/GP) SAND and GRAVEL, trace silt; brown to grey, angular; non-cohesive, moist, loose		0.00																
		(SM) - SILTY SAND, some clay, some gravel; grey to black, cobbles and boulders, contains organic matter (GLACIAL TILL); moist, compact		0.15																
1																				
2		- fine grained SAND; moist to wet																		
		END OF BOREHOLE Auger Refusal																		
3		Note(s): 1. Borehole dry upon completion of drilling. 2. Groundwater level measured at a depth of 3.59 m on October 12, 2023.																		
4																				
		Borehole continued on Record Drillhole BH23-07A		58.93																
5																				
6																				
7																				
8																				
9																				
10																				

Bentonite

Cuttings

Oct. 12, 2023

Bentonite

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447

RECORD OF DRILLHOLE: BH23-07A

SHEET 1 OF 1

LOCATION: N 5025621.99 ;E 439571.86

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Downing

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																FEATURES	PIEZOMETER																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA				WEATH- ERING INDEX	Diametral Point Load Index (MPa)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jzon																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
5		Cont'd from Record of Borehole BH23-07A Fresh, thinly to medium bedded, grey, fine to medium grained, faintly porous, strong LIMESTONE bedrock with slightly weathered to fresh, porous, fine grained, black shale		58.91 4.47	1																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				</

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

PROJECT: CA0008376.9447

LOCATION: N 5025623.80; E 439505.34

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-08

BORING DATE: August 25, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE			SAMPLES			HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected 100 200 300 400				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³					
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT					
								ND = Not Detected 100 200 300 400				Wp — W — Wi					
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		61.74											GR SA SI CL		
		FILL - (GP) sandy GRAVEL, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, very dense		0.00	1	SS	71	ND	⊕						37 41 (22)		
		(SM/GP) SILTY SAND and GRAVEL; dark to brown, contains organic matter, rootlets and rock fragments (GLACIAL TILL); moist, very dense		61.04 0.70	2	SS	42										
1				60.44	3	SS	50/ 0.08										
2		END OF BOREHOLE Auger Refusal		1.30													
3		Note(s):															
4		1. Borehole dry upon completion of drilling.															
5																	
6																	
7																	
8																	
9																	
10																	

DEPTH SCALE

1 : 50



LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

PROJECT: CA0008376.9447

LOCATION: N 5025685.05; E 439517.69

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

RECORD OF BOREHOLE: BH23-09

BORING DATE: August 24, 2023

DRILL RIG: CME 75

SHEET 1 OF 1

DATUM: Geodetic

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE		SAMPLES			HEADSPACE COMBUSTIBLE VAPOUR CONCENTRATIONS [PPM] ⊕				HYDRAULIC CONDUCTIVITY, k, cm/s				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
		DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ND = Not Detected				10 ⁻⁶ 10 ⁻⁵ 10 ⁻⁴ 10 ⁻³						
								HEADSPACE ORGANIC VAPOUR CONCENTRATIONS [PPM] □				WATER CONTENT PERCENT						
								ND = Not Detected				Wp — W — Wi						
0	Power Auger 204 mm Diam. (Hollow Stem)	GROUND SURFACE		59.68												GR SA SI CL		
		ASPHALT		59.68														
		FILL - (SP/GP) SAND and GRAVEL, trace silt; brown to grey, angular (PAVEMENT STRUCTURE); non-cohesive, moist, compact		59.43	1A		13	ND										
		FILL - (SP) SAND, some silt; brown to grey, angular; non-cohesive, moist, loose		0.25	1B			ND										
1					2		6	ND										
		TOPSOIL - (SM/ML) SILTY SAND to sandy SILT; black, contains rootlets and organic matter; non-cohesive		58.38	3A			ND										
		(SP/GP) SAND and GRAVEL, some silt; brown to light brown, angular (GLACIAL TILL); non-cohesive, moist, dense		1.30	3B		36	ND								41 39 (20)		
				1.40														
2			END OF BOREHOLE		57.81													
					1.87													
		Note(s):																
		1. Borehole dry upon completion of drilling.																
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		

DEPTH SCALE

1 : 50



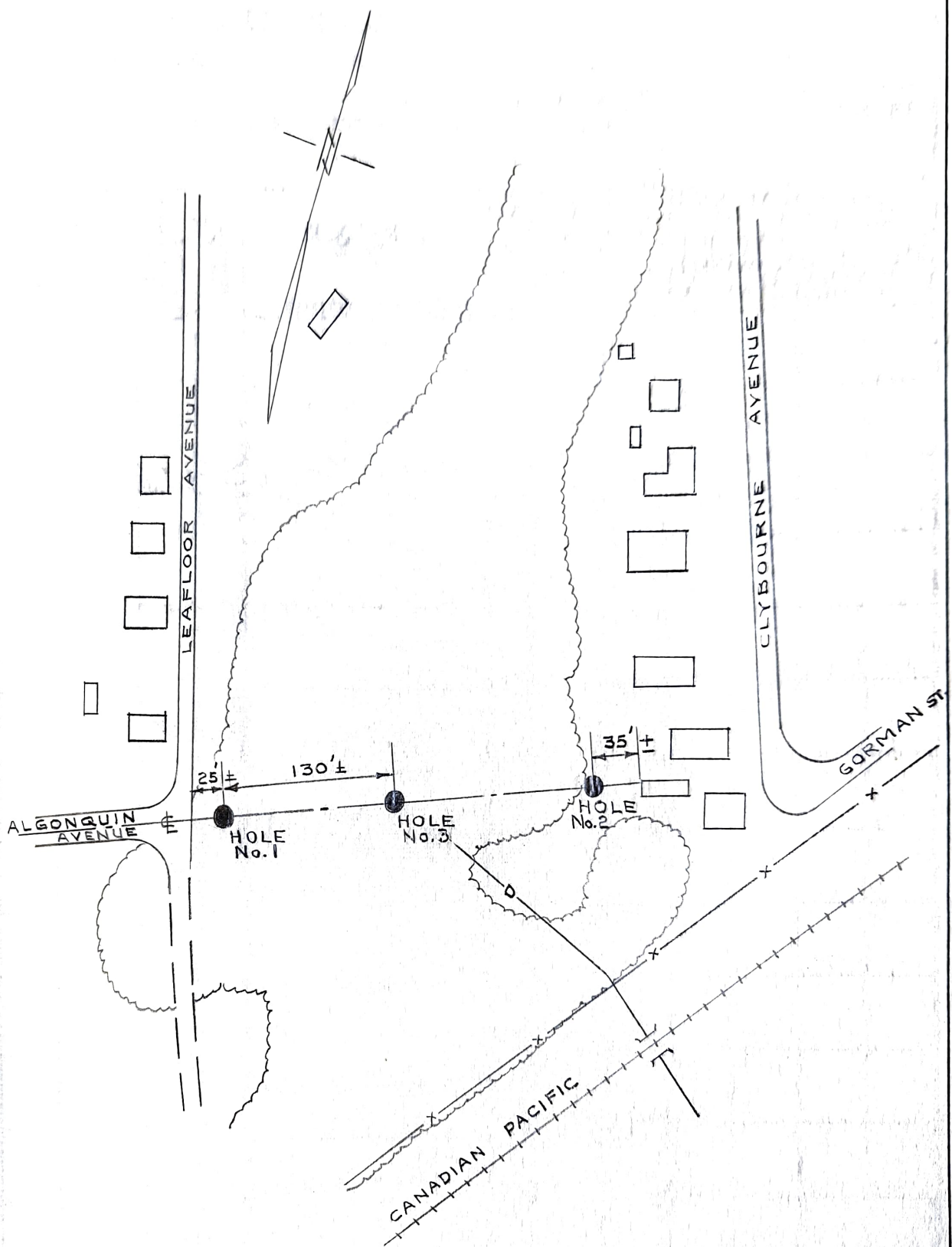
LOGGED: OB

CHECKED: AKP

GTA-BHS 005 S:\CLIENTS\THEIA PARTNERS\OTTAWA 30 CLEARY AVE\02 DATA\GINT\OTTAWA 30 CLEARY AVE.GPJ GAL-MIS.GDT 11/3/23

APPENDIX B

**Record of Borehole Sheet
(Previous Investigation)**



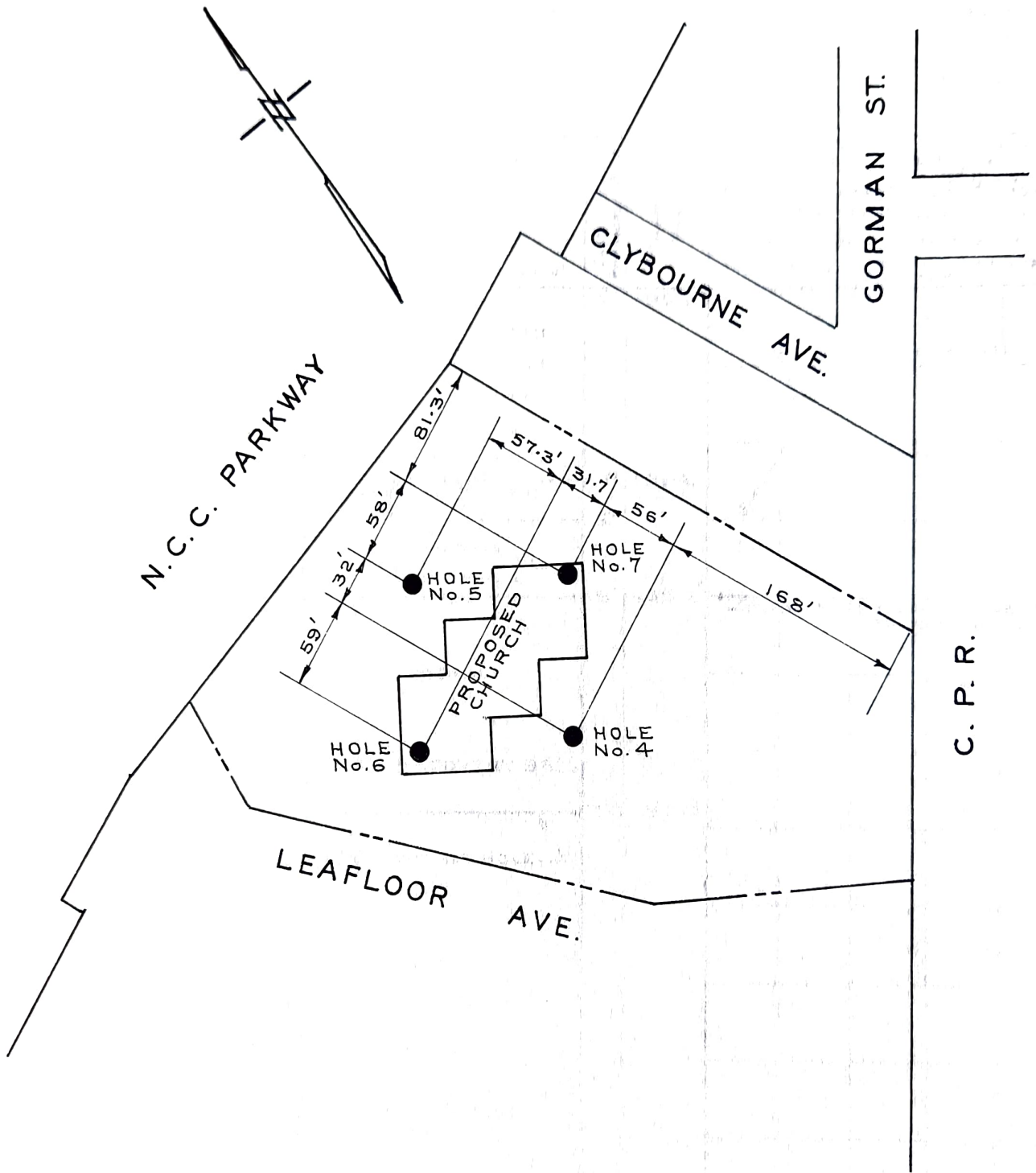
McROSTIE & ASSOCIATES LTD.	
CONSULTING ENGINEERS	
BOREHOLE LOCATIONS LEAFLOOR AVE. NEAR OTTAWA RIVER PARKWAY	
SCALE 1" = 100'	PLATE 1

LEAFLOOR NEAR
OTTAWA RIVER PARKWAY

HOLE NO.

3

						PROBING OR VANE TEST	
						L.B. HAMMER	NO CASING
						INCH DROP	INCH DIA. ROD
						BLOWS PER FOOT OR	SHEAR STRENGTH IN KIPS PER FT.
				GROUND SURFACE	DEPTH IN FEET	ELEVATION	
			FILL		0'		
			SAWDUST & WOOD WITH A LITTLE SILT				
	4	3.1A			3.8'		
		3.1B	LOOSE FINE SAND WITH A LITTLE MEDIUM SAND				
	6		LOOSE CLAYEY SILT WITH A LITTLE VERY FINE SAND A TRACE OF GRAVEL & A 1/2" LAYER OF FINE SAND		5'		
9 for 6"		3.2	MEDIUM DENSE CLAYEY SILT WITH A LITTLE VERY FINE SAND & A TRACE OF GRAVEL & A 1/2" LAYER OF FINE SAND		6'		
19 for 3"		3.3	DENSE TILL		7.5'		
					7.7'		
			ROCK				
			CORE RECOVERY 87%				
			BOTTOM OF HOLE		12.7'		
						WATER CONTENT 	
						% WATER CONTENT NATURAL LIMIT ○ LIQUID LIMIT □ PLASTIC LIMIT △	
						PLATE 4	



McROSTIE SETO GENEST

& ASSOCIATES LTD. - CONSULTING ENGINEERS

& ASSOCIÉS LTÉE - INGÉNIEURS CONSEILS

BOREHOLE LOCATIONS - POSITIONS DES FORAGES

LEAFLOOR NEAR
OTTAWA RIVER PARKWAY

SCALE
ÉCHELLE 1" = 100'

PLATE
PLAQUE 1

McROSTIE SETO GENEST

& ASSOCIATES LTD. & ASSOCIÉS LTÉE

CONSULTING ENGINEERS - INGÉNIEURS CONSEILS

OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

LEAFLOOR NEAR

OTTAWA RIVER PARKWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 196.2'
NIVEAU DU SOL (PROFONDEUR ZÉRO)

DATE SEPT. 30, 1965

HOLE FORAGE No.

4

NOTES JOB B.M. EL. 193.34' GEODETIC - TOP OF CONCRETE BASE FOR
CORNER POST OF N.C.C. FENCE AT N.E. CORNER OF PROPERTY.

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit pénétromètre K/Pd.2	Esai - Standard Penetration Blows/ft. - Coups/pd	No. Sample Echantillon	DESCRIPTION OF SOIL DU SOL	Depth in Feet Profondeur - Pied	Elevation Niveau	PROBING OR VANE TEST MARTEAU---HAMMER CHUTE LIBRE---DROP BLOWS/FOOT OR SHEAR STRENGTH K.S.F. COUPS/PIED OU RÉSISTANCE AU K/PD.2 CISAILEMENT	SONDAGE OU ESSAI AU MOULINET NO CASING SANS TUBAGE BARRE---DIA. ROD OVER-NIGHT WATER LEVEL BELOW 188.2'
				Ground Surface - Niveau du Sol	0'	196.2'		
				FILL				
			18 4-1	VERY FINE SAND WITH SOME SILT				
			9 6 4-2	A LITTLE CLAY & A TRACE OF WOOD & GRAVEL				
			18 29 4-3	MEDIUM DENSE SILTY TILL	5'	191.2'		
				DENSE SILTY TILL	6'			
			25 6 4-4	ROCK	8'	188.2'		
				CORE RECOVERY 98%	13.6'	182.6'		
				BOTTOM OF HOLE				

WATER CONTENT %TENEUR EN EAU	PLATE PLaque No.
NATURAL NATURELLE	2
LIQUID LIMIT LIMITE DE LIQUIDITÉ	
PLASTIC LIMIT LIMITE DE PLASTICITÉ	

R REMOULDED-REMANIÉ
CORE RECOVERY
CAROTTE RÉCUPÉRÉE

OTTAWA CANADA

PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

LEAFLOOR NEAR
OTTAWA RIVER PARKWAY

NOTES SEE PLATE No. 2

5

[illegible]

& ASSOCIATES LTD. & ASSOCIÉS LTÉE
CONSULTING ENGINEERS – INGÉNIEURS CONSEILS
OTTAWA CANADA

OTTAWA CANADA

SOIL PROFILE & TEST SUMMARIES
PROFIL SOUTERRAIN ET RÉSUMÉ DES ESSAIS

LEAFLOOR NEAR
OTTAWA RIVER PARKWAY

HOLE No.
FORAGE
& PIT 7

NOTES SEE PLATE No. 2

Compressive Strength K.S.F. Résistance à la Compression K/Pd.2	Small Scale Penetrometer K.S.F. Petit Pénétrmètre K/Pd.2	Essai - Standard Penetration Blows/Ft.-Coups /pd.	No. Sample Echantillon	DESCRIPTION OF SOIL DU SOL	Depth in Feet Profondeur - Pied	Elevation Niveau	PROBING OR VANE TEST		SONDAGE OU ESSAI AU MOULINET	
							MARTEAU----HAMMER	CHUTE LIBRE---DROP	NO CASING SANS TUBAGE	
							BLOWS/FOOT OR SHEAR STRENGTH K.S.F. COUPS/PIED OU RÉSISTANCE AU, K/PD.2 CISAILLEMENT		BARRE-----DIA. ROD	
				Ground Surface - Niveau du Sol						

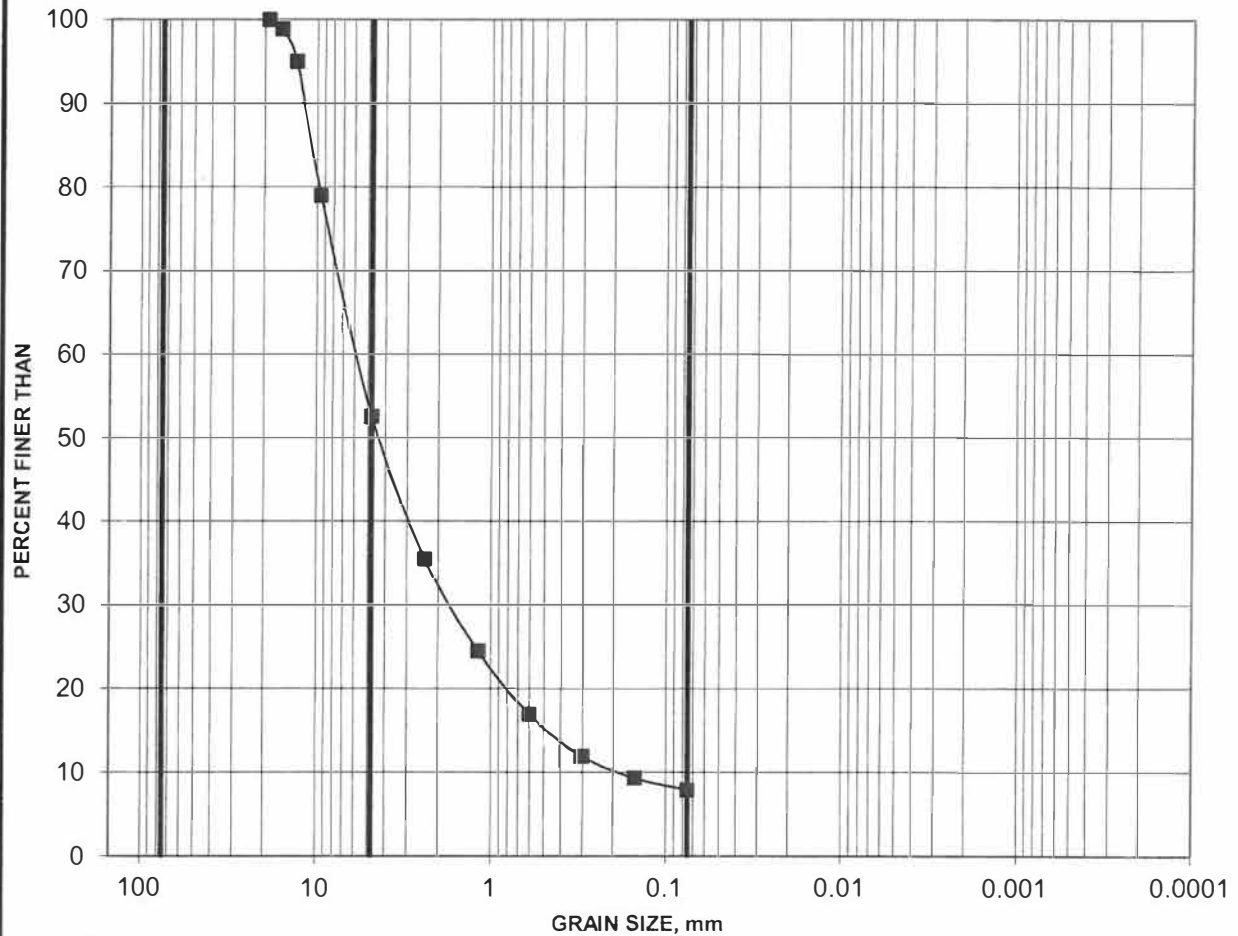
APPENDIX C

Laboratory Results

GRAIN SIZE DISTRIBUTION

FIGURE

SAND AND GRAVEL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIU	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ 23-03	1	0.00-0.61	47	45	8	

Project: CA0008376.9447

wsp

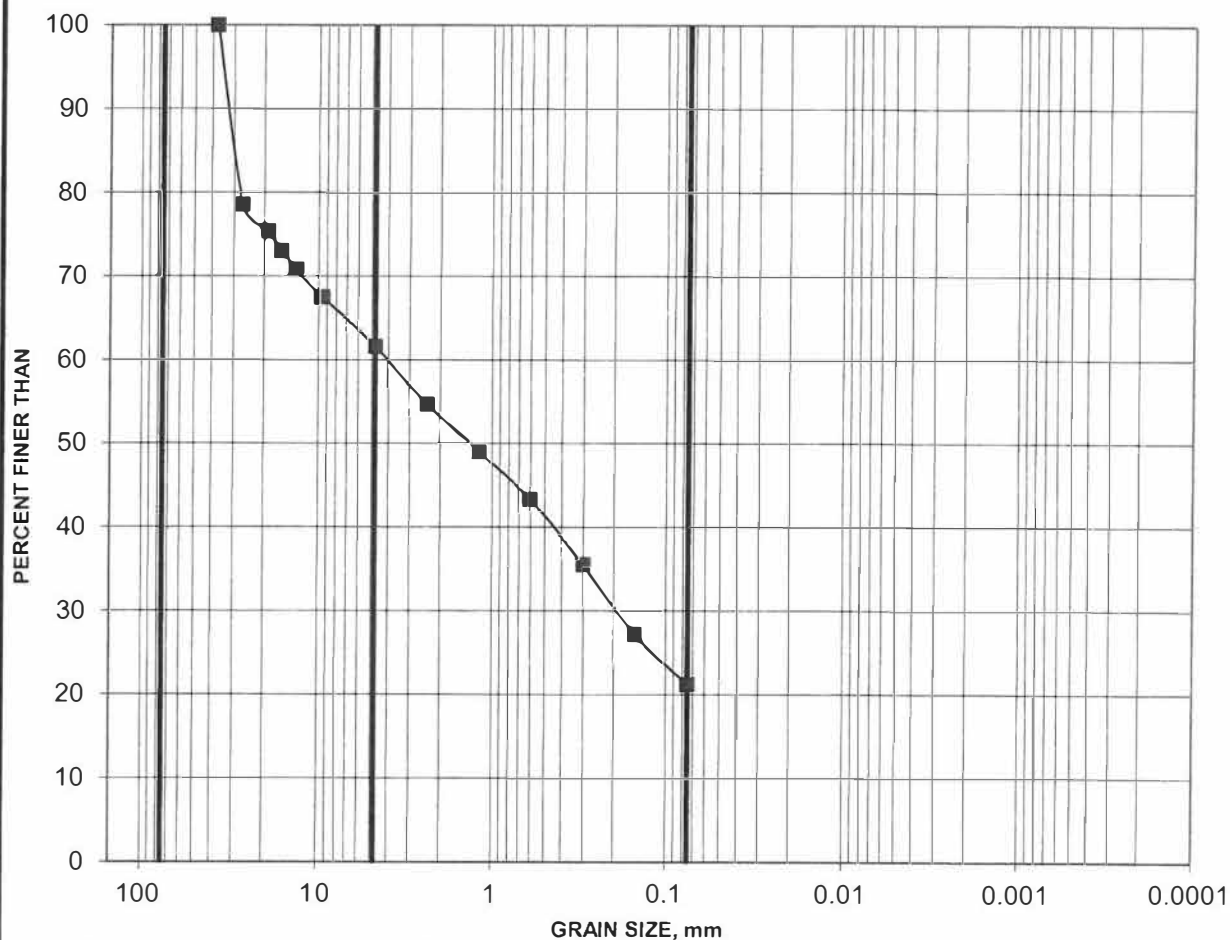
Created by: MI

Checked by: CW

GRAIN SIZE DISTRIBUTION

FIGURE

SILTY SAND AND GRAVEL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIU	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ 23-06	2	0.69-1.30	38	41	21	

Project: CA0008376.9447

wsp

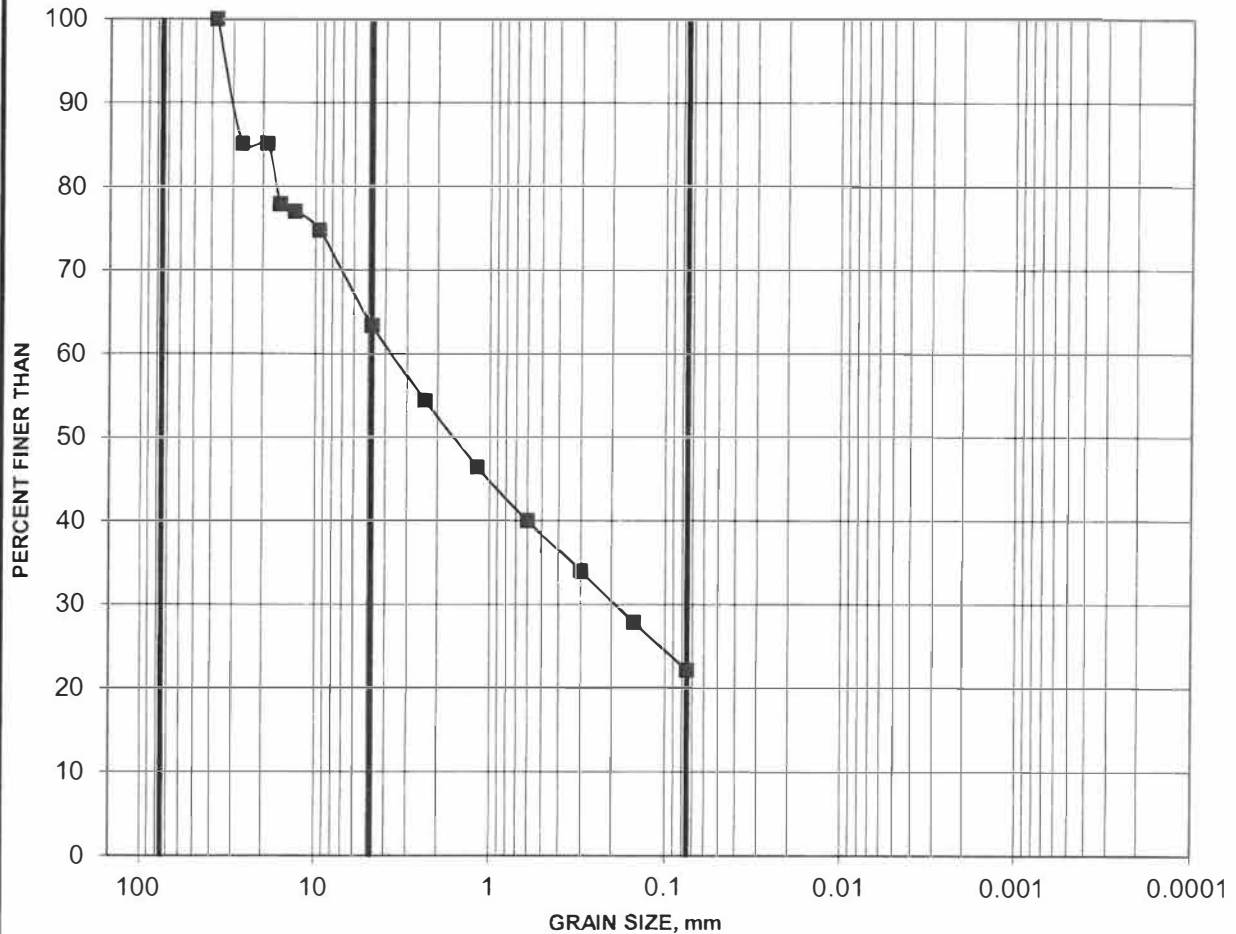
Created by: MI

Checked by: CW

GRAIN SIZE DISTRIBUTION

FIGURE

SILTY SAND AND GRAVEL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIU	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ 23-08	2	0.61-1.22	37	41	22	

Project: CA0008376.9447

wsp

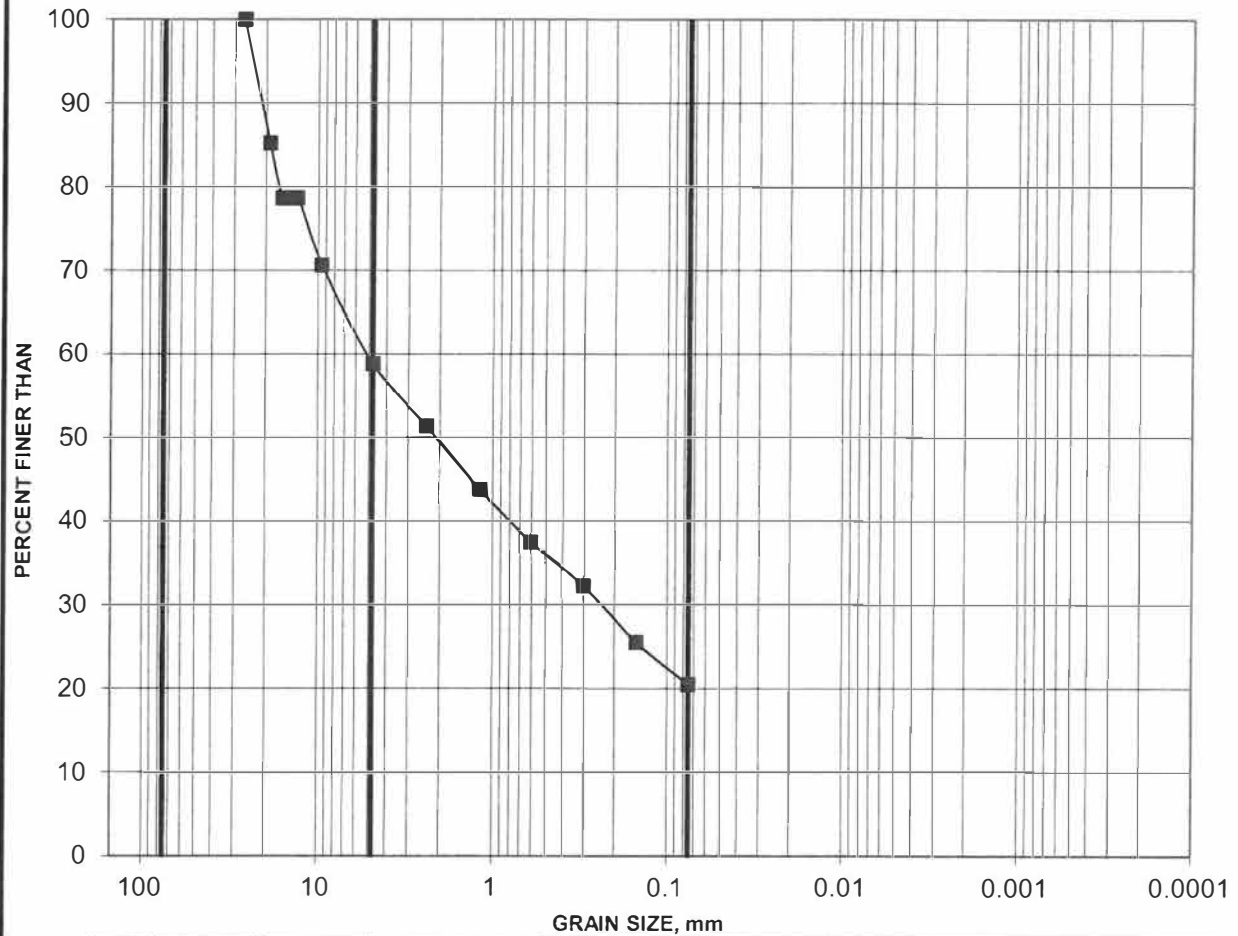
Created by: MI

Checked by: *MI*

GRAIN SIZE DISTRIBUTION

FIGURE

SILTY SAND AND GRAVEL



COBBLE SIZE	COARSE	FINE	COARSE	MEDIU	FINE	SILT AND CLAY
	GRAVEL SIZE		SAND SIZE			

Borehole	Sample	Depth (m)	Constituents (%)			
			Gravel	Sand	Silt	Clay
■ 23-09	3B	1.42-1.80	41	39	20	

Project: CA0008376.9447

wsp

Created by: MI

Checked by:

APPENDIX D

Basic Chemical Testing

Certificate of Analysis

Client: WSP Canada Inc (Ottawa)
1931 Robertson Road
Ottawa, Ontario
K2H 5B7
Attention: Mr. Arthur Kuitchoua Petke
PO#:
Invoice to: WSP Canada Inc.

Report Number: 3001153
Date Submitted: 2023-09-08
Date Reported: 2023-09-18
Project: CA0008376.9447
COC #: 910485

Page 1 of 3

Dear Arthur Kuitchoua Petke:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

Revised report to fix the ids as per clients request

APPROVAL:

Rebecca Koshy, Project Manager

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: <https://directory.cala.ca/>.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

Client: WSP Canada Inc (Ottawa)
1931 Robertson Road
Ottawa, Ontario
K2H 5B7
Attention: Mr. Arthur Kuitchoua Petke
PO#:
Invoice to: WSP Canada Inc.

Report Number: 3001153
Date Submitted: 2023-09-08
Date Reported: 2023-09-18
Project: CA0008376.9447
COC #: 910485

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	
Group	Analyte	MRL	Units	Guideline	1702367 Soil 2023-08-24 23-03 SA2	1702368 Soil 2023-08-24 23-05 SA3
Anions	Cl	0.002	%		0.005	0.006
	SO4	0.01	%		0.01	<0.01
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.29	0.23
	pH	2.00			8.07	8.76
	Resistivity	1	ohm-cm		3448	4348

Guideline = * = Guideline Exceedence

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client: WSP Canada Inc (Ottawa)
1931 Robertson Road
Ottawa, Ontario
K2H 5B7
Attention: Mr. Arthur Kuitchoua Petke
PO#:
Invoice to: WSP Canada Inc.

Report Number: 3001153
Date Submitted: 2023-09-08
Date Reported: 2023-09-18
Project: CA0008376.9447
COC #: 910485

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 448741 Analysis/Extraction Date 2023-09-13 Analyst IP Method Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	99	90-110
pH	6.38	99	90-110
Resistivity			
Run No 448900 Analysis/Extraction Date 2023-09-15 Analyst IP Method AG SOIL			
SO4	<0.01 %	94	70-130
Run No 448924 Analysis/Extraction Date 2023-09-15 Analyst AsA Method C CSA A23.2-4B			
Chloride	<0.002 %	106	90-110

Guideline = * = **Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX E

Rock Photos and Results of USC Testing

Top of Bedrock 0.86 mbgs

BH 23-01 (Dry)
Core Box 1 to 5 of 5



EOH: 7.90 mbgs



Geotechnical Investigation

Cleary Development

CA0008376.9447

Ottawa, Ontario

Project No.	CA0008376.9447
Drawn:	PAK
Date:	2023-08-29
Checked:	AKP
Review:	CH

Figure E1

BH 23-01 (Wet)
Core Box 1 to 5 of 5

Top of Bedrock 0.86 mbgs



EOH: 7.90 mbgs



Geotechnical Investigation

Cleary Development

CA0008376.9447

Ottawa, Ontario

Project No.	CA0008376.9447
Drawn:	PAK
Date:	2023-08-29
Checked:	AKP
Review:	CH

Figure E2

BH 23-05 (Dry)
Core Box 1 to 4 of 4

Top of Bedrock 1.93 m



EOH: 7.97 mbgs



Geotechnical Investigation

Cleary Development
CA0008376.9447
Ottawa, Ontario

Project No.	CA0008376.9447
Drawn:	PAK
Date:	2023-08-29
Checked:	AKP
Review:	CH

Figure E3

BH 23-05 (wet)
Core Box 1 to 4 of 4

Top of Bedrock 1.93m



EOH: 7.97 mbgs



Geotechnical Investigation

Cleary Development

CA0008376.9447

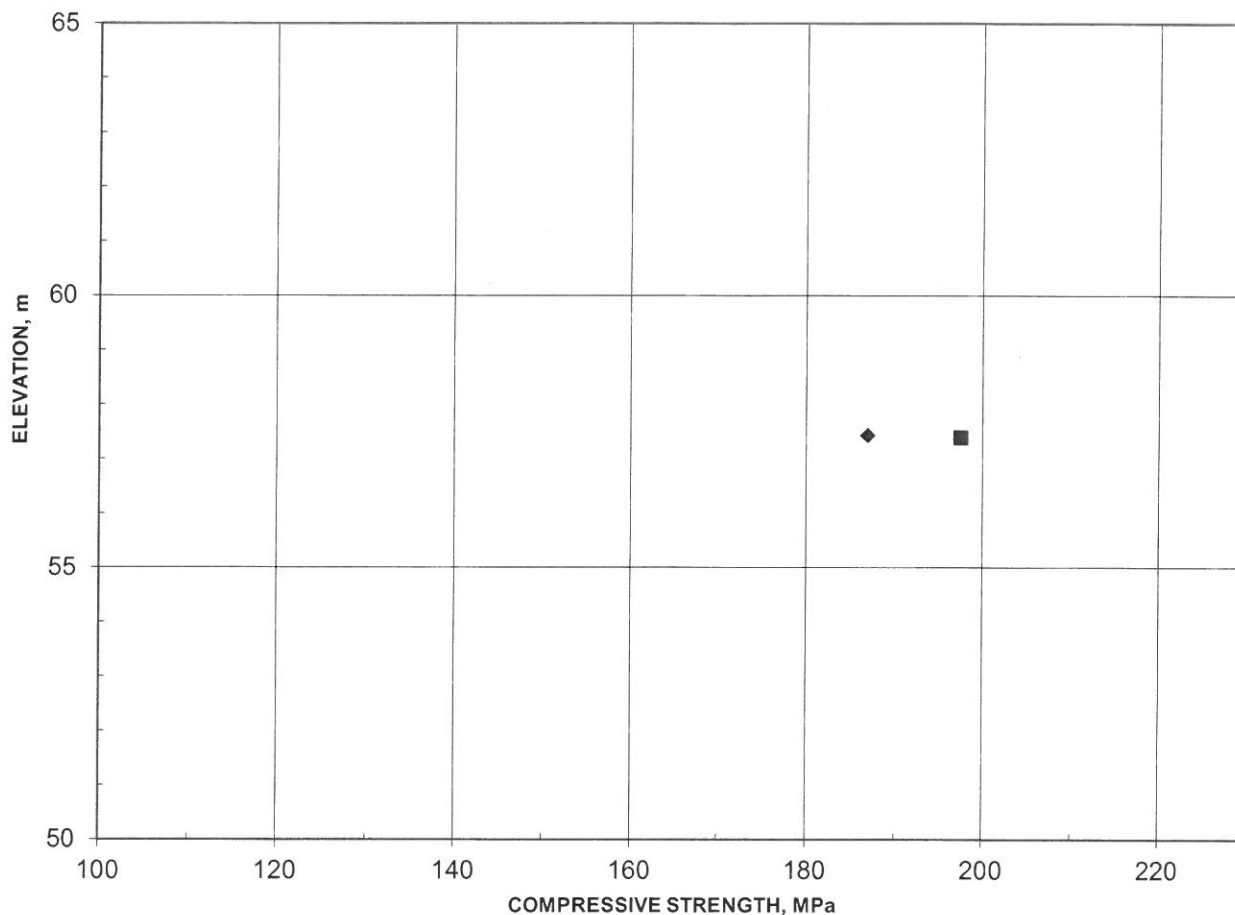
Ottawa, Ontario

Project No.	CA0008376.9447
Drawn:	PAK
Date:	2023-08-29
Checked:	AKP
Review:	CH

Figure E4

ASTM D7012 - Method C
UNIAXIAL UNCONFINED COMPRESSIVE STRENGTH OF ROCK CORE
SUMMARY OF LABORATORY TEST RESULTS

FIGURE



	Borehole	Depth (m)	L/D	Bulk Density (kg/m ³)	Lithology	UCS (MPa)	Failure Type
■	BH23-1 RC1	2.5	2.1	2619	Limestone	198	1
◆	BH23-5 RC1	4.7	2.4	2595	Limestone	187	1

Notes:

Failure Types

1. Well formed cones on both ends
2. Well formed cones on one end, vertical cracks through cap
3. Columnar vertical cracking through both ends
4. Diagonal fracture with no cracking through ends
5. Side fractures at top or bottom
6. Side fractures at both sides of top or bottom

Remarks

- Cores tested in vertical direction.
- Cores tested in air-dry condition.
- Time to failure > 2 and < 15 minutes.

Project: CA0008376.9447

wsp

Created by: MI

Checked by: CW

APPENDIX F

**Hydraulic Conductivity Testing and
Estimated Groundwater Inflow**

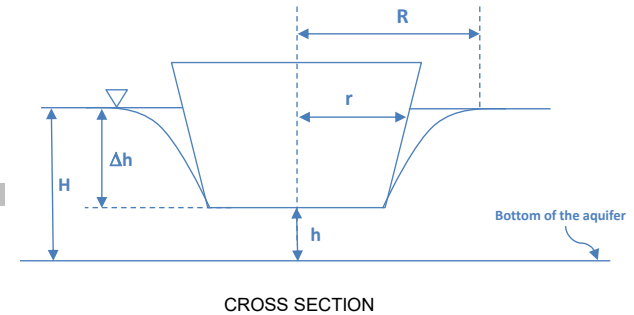
Inflow to Circular Excavation **Dupuit-Forchheimer Equation:**

$$Q = \left(\frac{\pi K (H^2 - h^2)}{\ln \left(\frac{R}{r} \right)} \right)$$

K (m/sec) **4E-06**
H (m) **7.7**
h (m) **1.0**
r (m) **32.7**

r - radius of pit
R - radius of influence

	Q (m³/s)	R	Rad of Inf. from edge	m³/day	L/day
Initial*	1.2E-02	34.7	2.0	1060.73	1,060,735
	5.1E-03	37.7	5.0	442.56	442,558
	2.7E-03	42.7	10.0	235.99	235,993
	1.9E-03	47.7	15.0	166.78	166,777
	1.5E-03	52.7	20.0	131.94	131,940
	1.3E-03	57.7	25.0	110.88	110,880
	1.1E-03	62.7	30.0	96.72	96,724
Steady-State**	9.1E-04	72.7	40.0	78.81	78,808
	7.9E-04	82.7	50.0	67.86	67,861
	7.0E-04	92.7	60.0	60.43	60,427
	6.4E-04	102.7	70.0	55.02	55,017
	5.9E-04	112.7	80.0	50.89	50,885
	5.5E-04	122.7	90.0	47.61	47,614
	5.2E-04	132.7	100.0	44.95	44,950
	4.2E-04	182.7	150.0	36.60	36,596
	3.7E-04	232.7	200.0	32.08	32,084
	3.4E-04	282.7	250.0	29.19	29,189



Sichart and Kyrieleis Equation: $R = \frac{3000 \Delta h \sqrt{K}}{\text{Radius of Influence (m)}}$

Rainfall Amount - Based on a 72 mm precipitation event in 24 hours **with a return of 10 years**

Excavation Area (m²)	3361.1
10 year Rainfall event (m)	0.072
Max Vol Precipitation (L)	241,999

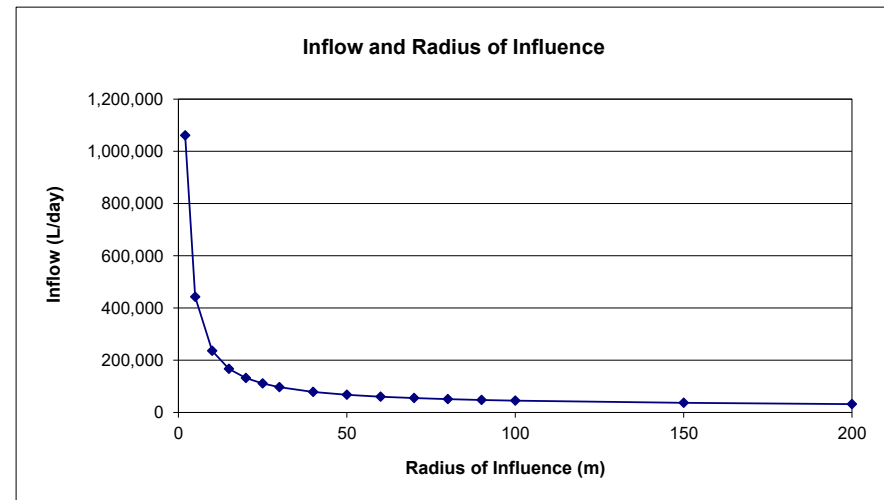
Notes

L - litres

m - metres

Initial*: Potential worst-case inflow rate when excavation is initially rapidly dewatered

Steady-State**: Estimated steady state inflow rate if dewatering extends to R



HVORSLEV SLUG TEST ANALYSIS **FALLING HEAD TEST MW26-01**

INTERVAL (metres below ground surface)

Top of Interval = 2.49
Bottom of Interval = 8.59

$$K = \frac{r_c^2}{2L_e} \ln \left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e} \right)^2} \right] \left[\frac{\ln \left(\frac{h_1}{h_2} \right)}{(t_2 - t_1)} \right] \quad \text{where K = (m/sec)}$$

where:

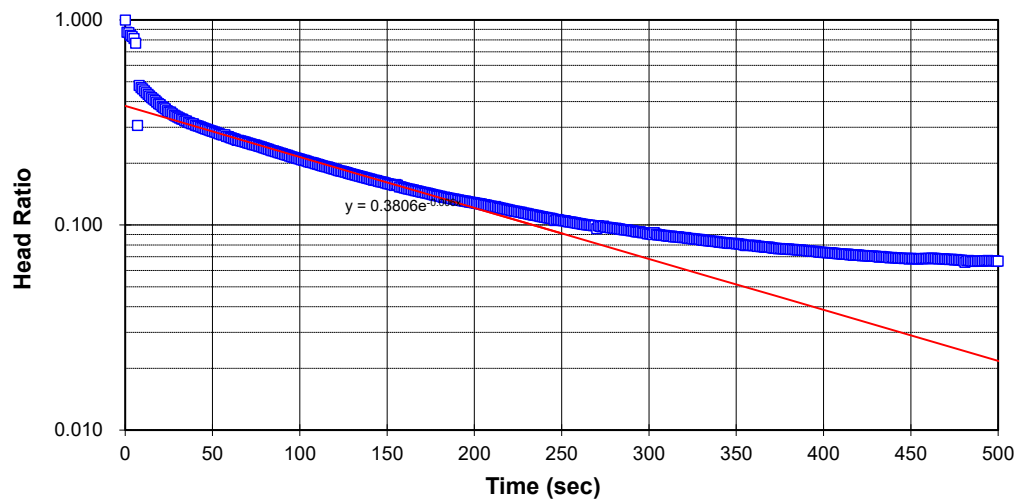
- r_c = casing radius (metres)
- R_e = filter pack radius (metres)
- L_e = length of screened interval (metres)
- t = time (seconds)
- h_t = head at time t (metres)

INPUT PARAMETERS

r_c = 2.5E-02
 R_e = 4.8E-02
 L_e = 1.5
 t_1 = 53
 t_2 = 251
 h_1/h_0 = 0.28
 h_2/h_0 = 0.09

RESULTS

K= 4E-06 m/sec
K= 4E-04 cm/sec



Project Name: Cleary Development
 Project No.: CA0008376.9447
 Test Date: 01-May-26

Analysis By: EL
 Checked By: CAMC
 Analysis Date: 12-May-26

WSP Canada Inc.

HVORSLEV SLUG TEST ANALYSIS **FALLING HEAD TEST MW26-02**

INTERVAL (metres below ground surface)

Top of Interval = 1.42
Bottom of Interval = 6.91

$$K = \frac{r_c^2}{2L_e} \ln \left[\frac{L_e}{2R_e} + \sqrt{1 + \left(\frac{L_e}{2R_e} \right)^2} \right] \left[\frac{\ln \left(\frac{h_1}{h_2} \right)}{(t_2 - t_1)} \right] \quad \text{where K = (m/sec)}$$

where:

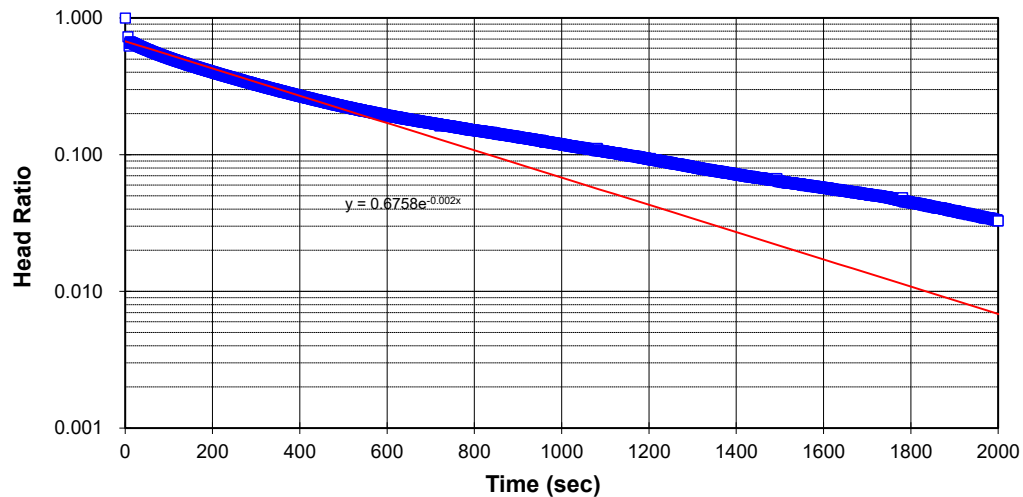
- r_c = casing radius (metres)
- R_e = filter pack radius (metres)
- L_e = length of screened interval (metres)
- t = time (seconds)
- h_t = head at time t (metres)

INPUT PARAMETERS

r_c = 2.5E-02
 R_e = 4.8E-02
 L_e = 1.5
 t_1 = 21
 t_2 = 411
 h_1/h_0 = 0.64
 h_2/h_0 = 0.26

RESULTS

K= 2E-06 m/sec
K= 2E-04 cm/sec



Project Name: 40 Cleary Development
 Project No.: CA0008376.9447
 Test Date: 08-May-26

Analysis By: EL
 Checked By: CAMC
 Analysis Date: 12-May-26

WSP Canada Inc.

APPENDIX G

**Multichannel Analysis of Surface Waves (MASW)
Technical Memorandum**



TECHNICAL MEMORANDUM

DATE June 04, 2026

Project No. CA0008376.9447

FROM Alex Bilson Darko, Jonathan Crawford

EMAIL Alex.Bilson.Darko@wsp.com;
Jonathan.Crawford@wsp.com

MASW SURVEY RESULTS – 30 CLEARY AVE OTTAWA

This technical memorandum presents the processing and results of the Multichannel Analysis of Surface-Waves (MASW) testing performed for the purpose of Seismic Site Classification at 30 Cleary Ave Ottawa, Ontario. The geophysical testing was performed by WSP personnel on April 17, 2026, along the survey lines shown in Figure 1, below.



Figure 1: Survey Line Locations in red.

Methodology

The MASW method measures variations in surface-wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface-waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium surface-waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that wavelength of surface-wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledgehammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface-wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.

Field Work

The field work was conducted on April 17, 2026, by personnel from the WSP Mississauga office. Data was acquired using a Geometrics Geode 24 channel seismograph and 24 low frequency (4.5 Hz) geophones.

For the two MASW lines, the geophones were laid out at 1 metre intervals. An 8-kilogram (kg) sledgehammer and 40 kg seismic weight drop were used as seismic sources for this investigation. Seismic records were collected with seismic sources located 5 and 10 m, from and collinear to the geophone array. An example of an active seismic record collected at the site is shown in Figure 2.

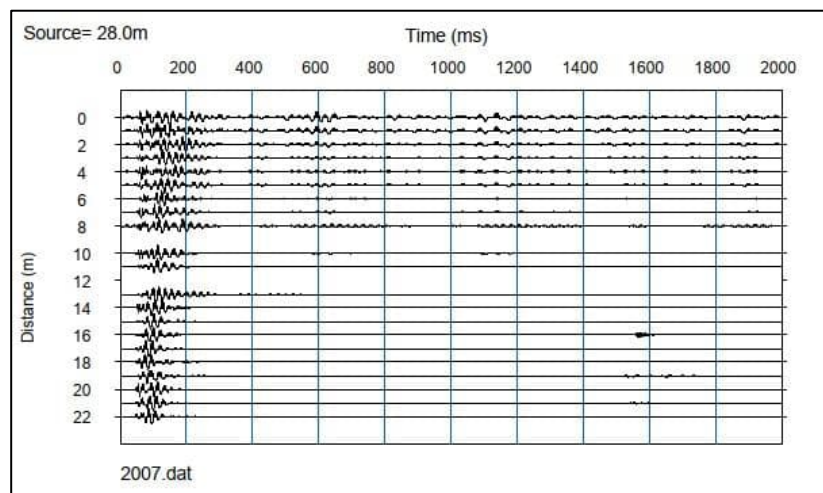


Figure 2: Typical seismic record collected for the MASW Line

Data Processing

Processing of the MASW test results consisted of the following main steps:

- Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
- Calculation of the phase for each frequency component;
- Linear regression to calculate phase velocity for each frequency component;
- Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r^2) between the data and the linear regression best fit line used to calculate phase velocity;
- Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and
- Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown in Figure 3. Shear-wave velocity (V_s) profiles were generated through inverse modelling to best fit the calculated fundamental mode dispersion curves.

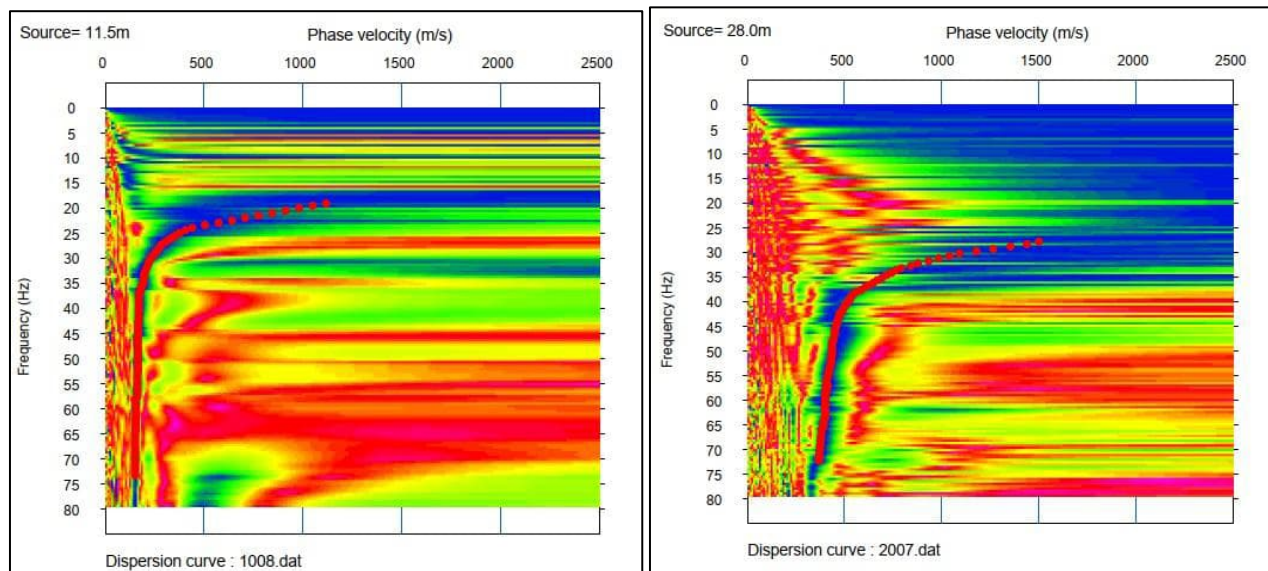


Figure 3: Example MASW Dispersion Curve Picks (red dots) for MASW Line 1 (left panel) and Line 2 (right panel).

The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 20 Hz for the MASW Lines.

Results

The MASW test results are presented in Figure 4 as the calculated shear-wave velocity profile measured from the MASW Lines. There is good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 5%.

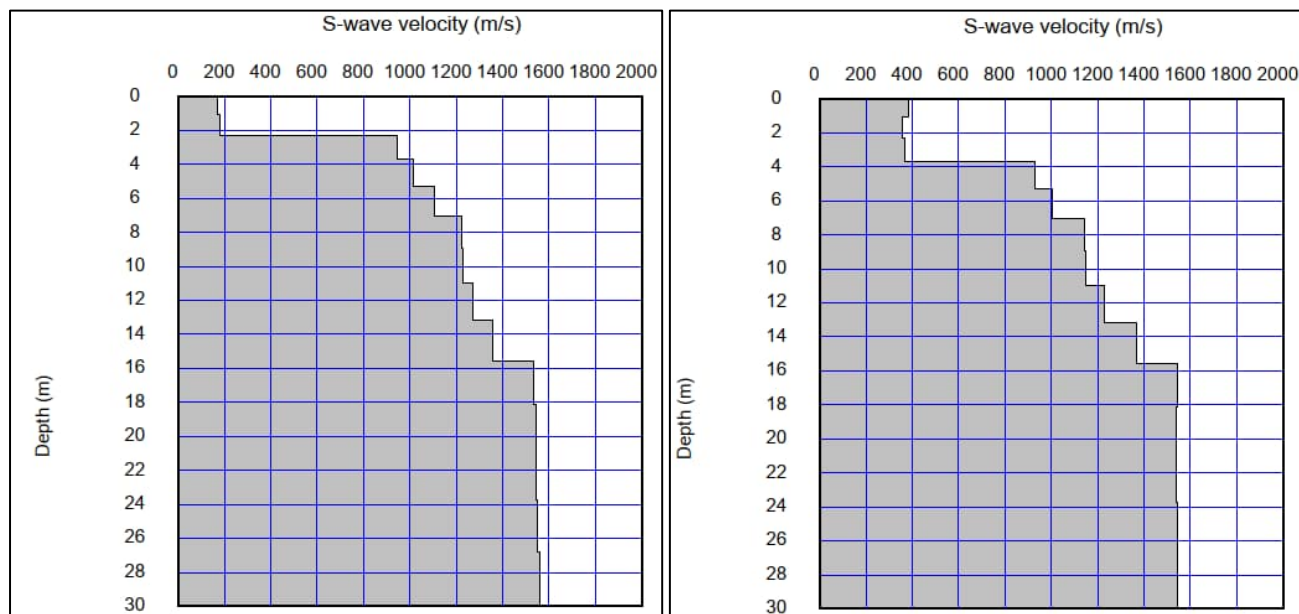


Figure 4: MASW Modelled Shear-Wave Velocity Depth profiles for MASW Line 1 (left panel) and Line 2 (right panel).

Table 1: Shear-Wave Velocity Profile for the MASW Line 1

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0	1.1	1.1	169	0.0065015
1.1	2.3	1.2	177	0.0067886
2.3	3.7	1.4	944	0.0014824
3.7	5.3	1.6	1018	0.0015722
5.3	7.0	1.7	1106	0.0015370
7.0	8.9	1.9	1225	0.0015513
8.9	11.0	2.1	1230	0.0017076
11.0	13.2	2.2	1270	0.0017320
13.2	15.6	2.4	1356	0.0017698
15.6	18.1	2.5	1530	0.0016337
18.1	20.9	2.8	1543	0.0018147
20.9	23.7	2.8	1543	0.0018147
23.7	26.8	3.1	1551	0.0019993
26.8	30.0	3.2	1563	0.0020472
Vs Average to 30 mbgs (m/s)				884

Table 2: Shear-Wave Velocity Profile for the MASW Line 2

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0	1.1	1.1	386	0.0028533
1.1	2.3	1.2	358	0.0033500
2.3	3.7	1.4	369	0.0037894
3.7	5.3	1.6	932	0.0017171
5.3	7.0	1.7	1005	0.0016915
7.0	8.9	1.9	1144	0.0016609
8.9	11.0	2.1	1149	0.0018277
11.0	13.2	2.2	1230	0.0017889
13.2	15.6	2.4	1368	0.0017543
15.6	18.1	2.5	1544	0.0016195
18.1	20.9	2.8	1540	0.0018179
20.9	23.7	2.8	1539	0.0018194
23.7	26.8	3.1	1541	0.0020116
26.8	30.0	3.2	1542	0.0020748
Vs Average to 30 mbgs (m/s)				1008

To calculate the average shear-wave velocity as required by Seismic Site Classification, the results were modelled to 30 metres below ground surface (mbgs).

The time-averaged shear-wave velocity (V_{s30}) for the MASW Lines were found to be 884 m/s and 1008 m/s for Line 1 and 2 respectively (Table 1 and 2). Based on the National Building Code of Canada (2020), these V_{s30} values correspond to a seismic site class B.

The seismic site class provided is based solely on the average shear wave velocity derived from this study. If there are site specific conditions present, such as liquefiable soils, clay layers with certain properties that have thicknesses of greater than 3 m, etc. these could change this seismic site classification. For more information on these potential conditions the reader should review section 4.1.8.4 of the National Building Code of Canada (2020).

Limitations of Use

The geophysical interpretation presented in this report is based on the use of geophysical surveying techniques. As with any geophysical method, interpretation presented in this report should be confirmed by intrusive methods (boreholes, test pits, etc.).

Assumptions made in the geophysical interpretation have been stated, where applicable, throughout the report.

This geophysical survey was carried out in a manner consistent with the level of care and skill normally exercised by other members of the engineering and science professions currently practising under similar conditions, subject to the time limits and financial and physical constraints applicable to the services provided. This report provides a professional opinion and therefore no warranty is either expressed, implied, or made as to the conclusions, advice, and recommendations offered.

Any use of the information within this report made by a third party, or any reliance on, or decisions to be made based on it, are the sole responsibility of such third parties. WSP accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions taken based on this letter.

Closure

We trust that this technical memorandum meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience.

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