PRI ENGINEERING

Geotechnical Investigation Report - Final

25-578 Waverly Public School
Oshawa, ON

Prepared for Durham District School Board

100 Waverly St
Oshawa, ON L1J 5V1

PRI ENGINEERING

February 24, 2026

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Emailed to: *stewart.osinga@ddsb.ca*

Subject: **Geotechnical Investigation Report**
Waverly Public School, Oshawa, ON
PRI Proposal No.: 25-578

Dear Mr. Osinga:

PRI Engineering Corp. (PRI) is pleased to submit the following geotechnical investigation report, describing subsurface conditions and providing geotechnical recommendations for the design and construction of the proposed elevator addition at Waverly Public School, located at 100 Waverly St, Oshawa, Ontario. It is understood that the proposed work includes the installation of a new elevator providing access to the existing institutional structure.

This report presents the results of the subsurface investigation for the subject site, which was completed on January 2nd, 2026, and includes our comments and recommendations as they relate to the existing foundation conditions, design and construction. Attached are the following: site plan noting borehole locations, borehole logs, and laboratory test results.

We trust that this is straightforward and meets your present requirements. Please contact us if you have any questions.

Yours truly,
PRI Engineering Corp.

VGledhill

Vikki Gledhill, P. Eng.
Director, Engineering

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List of Acronyms and Abbreviations

ASTM	American Society for Testing and Materials
ANSI	American National Standards Institute
AWWA	American Water Works Association
CCIL	Canadian Council of Independent Laboratories
CFEM	Canadian Foundation Engineering Manual
CPT	Cone Penetration Test
CSA	Canadian Standards Association
mBGS	Metres Below Ground Surface
mbeg	Metres Below Existing Grade
OBC	Ontario Building Code
OHS	Occupational Health and Safety
PRI	PRI Engineering Corp.
SPMDD	Standard Proctor Maximum Dry Density
SPT	Standard Penetration Test
USCS	Unified Soil Classification System

1 Introduction

As requested by the Durham District School Board (DDSB), PRI Engineering Corp. (PRI) is pleased to submit the following Geotechnical Investigation Report for the proposed elevator installation and associated interior renovations at Waverly Public School in Oshawa, Ontario. The purpose of this report is to characterize the subsurface conditions in the vicinity of the proposed construction to support the design of the proposed elevator, as well as to provide geotechnical considerations related to earthworks and foundation construction adjacent to the existing structure for the planned construction.

The project site, located at 100 Waverly St, in Oshawa (the "Site"), generally comprises asphalt parking areas and access roads, paved school yards, existing institutional buildings, landscaped zones, grassed play areas, and limited tree coverage. The existing school includes an original structure constructed in 1969 and three additions to the north, south and west constructed in 1991 and 1971, respectively.

The Site location, approximate property boundaries and the locations of the boreholes completed as part of this investigation are shown on **Figure 1**.

A summary of the reviewed background information is described in **Section 2** of this report. The field program procedures and associated laboratory program are summarized in **Section 3**. The subsurface profile and conditions are outlined in **Section 4**, with geotechnical recommendations and considerations related to the proposed foundations summarized in **Section 6**.

2 Background Information

Prior to mobilization to the Site, PRI reviewed the following references as part of the background information review:

- *Site Plan Waverly PS – 153,* prepared by *DDSB,* dated *January 10, 2022*;
- Water well records from the Ministry of the Environment, Conservation and Parks, Waterwell Database;
- *Bedrock Geology of Ontario* and *Surficial Geology of Ontario KML* Data Files, *Ontario Geological Survey, and*
- Government database for *Physiography of Southern Ontario.*

PRI reviewed available and published local geology data for the site. Based on the surficial geology data from Geology Ontario, the surficial geology consists of coarse-textured glaciolacustrine deposits, which are described as sand, gravel, minor silt and clay, which are considered foreshore and basinal deposits. Based on the bedrock geology data from Geology Ontario, the Site lies within the Georgian Bay Formation, Blue Mountain Formation, Billings Formation, Collingwood Member, and Eastview Member, which are generally composed of shale, limestone, dolostone, and siltstone. According to physiography data of southern Ontario, the physiographic landforms consist of the Till Plains.

Five (5) historical groundwater well records available within a 1.2 km vicinity of the Site were reviewed for this report. Based on water well records, clay, silt and sand overburden are noted to be predominant at all well locations. No bedrock was encountered in any of the five (5) well locations up to 25 meters below the ground surface (mBGS). Static water levels were reported at a depth of 12.2 mBGS at one (1) well location, 4601060. The pertinent information from the reviewed water well records is summarized in **Table 1.**

Table 1: Summary of Water Well Records

Well Records	Distance from Site (km)	GPS Coordinates	Static Water Level (mBGS)	Recorded Depth (mBGS)	Lithology
4601060	1.15	43.88249, -78.89853	12.2	0 to 0.3	Oven Clay
				0.3 to 4.9	Brown Clay
				4.9 to 25	Grey Clay & Boulders, Hard Pan
7050775	0.28	43.88839, -78.89077	N/A	0 to 2	Brown Topsoil
				2 to 10.2	Brown Clay
				10.2 to 16	Grey Clay & Boulders, Hard Pan
7128387	0.41	43.88943, -78.88219	N/A	0 to 13	Brown Sand
				13 to 20	Grey Silt
7143503	0.20	43.88850, -78.88978	N/A	0 to 0.2	Black Asphalt
				0.2 to 0.3	Brown Granular
				0.3 to 3	Brown Sandy Silt Till
				3 to 6.1	Brown Clay Silt
7327647	0.33	43.88941, -78.89144	N/A	0 to 0.3	Brown Sand
				0.3 to 3	Brown Sand
				3 to 4.6	Grey Silt
				4.6 to 6.1	Grey Sand

3 Geotechnical Investigation

3.1 Borehole Field Investigation Program

Prior to the field investigation, underground utility locates, including water, electrical, sewer, gas, telephone, cable, etc., were completed using Ontario One-call services and a private locator. Investigation locations were finalized in the field based on utility clearance and other obstructions (i.e. trees, overhead lines, vehicles, equipment, etc.) observed at the time of the investigation.

A borehole program was carried out on January 2, 2026. One (1) borehole, designated as BH26-01, was advanced at an accessible location to a sampling termination depth of approximately 15.8 mBGS. The borehole location was advanced at a pre-selected location on the west side of the existing building, using a track-mounted drill rig equipped with a 102 mm Outer Diameter (O.D.) solid stem auger and split spoons operating under full-time supervision of PRI and in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). The result of SPT sampling in terms of N-values is referred to as consistency for cohesive soils and relative density for non-cohesive materials.

Following completion of soil sampling at 15.8 mBGS, a Dynamic Cone Penetration Test (DCPT) was advanced within the borehole to a depth of approximately 18.2 mBGS, at which point refusal was encountered.

A qualified PRI geotechnical engineering technician supervised the drilling and logged and sampled the boreholes in accordance with industry standards. Subsurface conditions were logged in the field in accordance with PRI geotechnical protocols. Recovered soil samples were inspected and logged in the field by PRI personnel using visual and tactile methods. Soil samples were placed in moisture-proof containers for transportation to the laboratory for review and selected testing. Subsurface conditions, including groundwater seepage, were logged prior to backfilling.

A Borehole Location Plan is provided in **Figure 1.** Borehole logs are included as **Appendix A**. Borehole location information and depth details are summarized in **Table 2**.

Table 2: Borehole GPS Coordinates and Termination Depths

Borehole ID	GPS Coordinates		Termination Depth (mBGS)
	Latitude	Longitude	
BH26-01/ MW26-01	43.888574	-78.887597	18.2

3.2 Monitoring Well Installation

One (1) groundwater monitoring well was installed at borehole BH26-01 and designated as MW26-01 to evaluate static groundwater levels. The monitoring well was constructed using a 50 mm Inner Diameter schedule-40 Polyvinyl Chloride machine-slotted screen and riser pipe, packed with well sand, sealed with bentonite and backfilled with cuttings to grade. The monitoring well may be used during construction upon written agreement with PRI to confirm groundwater elevations for dewatering activities, but should be decommissioned thereafter, according to the Regulations. Monitoring well information is summarized in the borehole logs provided in **Appendix A**.

3.3 Laboratory Testing Program

Soil samples obtained from the field investigation program were recovered and retained in moisture-proof containers for further review, selected testing, and storage. Selected samples were submitted to a Canadian Certified Independent Laboratories (CCIL) certified laboratory, and SGS Canada Inc., for the tests summarized in **Table 3.**

Table 3: Laboratory Test Quantities and Reference Standards

Laboratory Test	Reference Standard	Number of Tests
Natural Moisture Content	ASTM D2216-98	13
Particle Size Distribution Analysis	ASTM D422 and ASTM D6913	2
Atterberg Limit	ASTM 4318	1
Corrosivity Analysis	Various Standards	1

Results from the Natural Moisture Content Analysis are summarized on the borehole logs with Particle Size Distribution Curves and Atterberg Limit Results provided as **Appendix B**. A summary of corrosivity analyses as per the ANSI/AWWA rating system is discussed in **Section 4**. The Certificate of Analysis and the ANSI/AWWA rating system are provided in **Appendix C**.

4 Subsurface Conditions

The inferred subsurface profiles are based on the borehole logs from the field investigation program. While we believe conditions are representative of actual site conditions, if findings during construction deviate from those encountered at the completed borehole, PRI should be consulted to revise our recommendations based on actual conditions at the time of construction. The following are the specific subsurface conditions encountered at borehole locations.

4.1 Pavement Structure

4.1.1 Surficial Pavement

A surficial layer of asphalt pavement was encountered at borehole BH26-01, with an approximate thickness of 100 mm.

4.1.2 Granular Fill/Fill

A layer of brown sand and gravel / silty sand granular fill was encountered at borehole BH26-01 directly underlying the surficial asphalt layer, with an approximate thickness of 450 mm. This material is inferred to be imported to the site for the pavement structure and was generally described as moist. Based on an SPT N-value of 17 blows per 305 mm of penetration, the fill has a compact relative density.

4.2 Topsoil

A layer of topsoil was encountered underlying the granular fill and was measured to be approximately 1.1 m in thickness. Assessment of organic matter content or other topsoil quality tests was beyond the scope of this current study.

4.3 Sand

Grey to brown sand was encountered underlying the topsoil, extending to a depth of 3.3 mBGS. The material contained trace amounts of silt and was described as saturated based on laboratory moisture contents of 19% to 20%. Based on SPT ‘N’ values ranging from 16 to 25, the sand has a compact relative density.

4.4 Clayey Silt

Grey clayey silt was encountered underlying the sand, extending to a depth of 15.2 mBGS. The clayey still contained trace amounts of sand and was described as about plastic limit (APL) to wetter than the plastic limit (WTPL) based on laboratory moisture contents of 12% to 38%. Based on SPT ‘N’ values of 0 to 7 blows per 305 mm of penetration, the clay silt had a very soft to firm consistency. Based on FVT's results, the in-situ undrained shear strength (s_u) of the soft clayey silt material ranges between 20 kPa and 75 kPa, with remoulded FVT values ranging from 7 kPa to 16 kPa and indicates sensitivities of approximately 3.2 to 6.6.

One (1) laboratory particle size distribution analysis was conducted on sampled clayey silt. Test results are provided in **Appendix B** and summarized in **Table 4**, as per USCS.

Table 4: Summary of Laboratory Particle Size Analyses –Clayey Silt

Borehole ID	Sample No.	Depth (mBGS)	Gravel* (%)	Sand** (%)	Silt *** (%)	Clay**** (%)
BH26-01	SS7	5.3 - 5.8	0	1	56	43

*Material passing 3-inch sieve opening and retained by No. 4 sieve.
**Material passing No. 4 sieve and retained by No. 200 sieve.
***Material passing No. 200 sieve and greater than 0.002 mm (based on hydrometer results).
****Material smaller than 0.002 mm (based on hydrometer results).

One (1) Atterberg Limit test was conducted on a selected sample of the clayey silt. The test results are provided in **Appendix B** and summarized in **Table 5**, as per USCS.

Table 5: Summary of Atterberg Limits Test —Clayey Silt

Borehole ID	Sample No.	Depth (mBGS)	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification
BH26-01	SS7	5.3 - 5.8	25	14	11	CL

4.5 Silty Sand / Silt and Sand

Layers of grey to brown silty sand/silt and sand were encountered within and below the clayey silt, at depths of 10.9 mBGS, and extending to the sampling termination depth of 15.8 mBGS. The material was described as containing some trace amounts of clay and trace amounts of gravel. The material was described as moist to saturated based on laboratory moisture contents of 9% to 17% and as having a loose relative density based on SPT 'N' values of 8 to 9 blows per 305 mm of penetration.

One (1) laboratory particle size distribution analysis was conducted on sampled silty sand/silt and sand. Test results are provided in **Appendix B** and summarized in **Table 6**, as per USCS.

Table 6: Summary of Laboratory Particle Size Analyses –Silty Sand / Silt and Sand

Borehole ID	Sample No.	Depth (mBGS)	Gravel* (%)	Sand** (%)	Silt *** (%)	Clay**** (%)
BH26-01	SS13	15.2 - 15.8	7	40	40	14

*Material passing 3-inch sieve opening and retained by No. 4 sieve.
**Material passing No. 4 sieve and retained by No. 200 sieve.
***Material passing No. 200 sieve and greater than 0.002 mm (based on hydrometer results).
****Material smaller than 0.002 mm (based on hydrometer results).

4.6 Bedrock and Other Obstructions

The borehole was advanced to DCP refusal at a depth of approximately 18.2 mBGS. Refusal was encountered on material exhibiting drilling resistance consistent with very dense soil or bedrock.

4.7 Groundwater and Borehole Stability Observations

The borehole BH26-01 remained open and dry upon completion of drilling. It should be noted that groundwater levels may vary and are subject to seasonal fluctuations in response to climatic weather events.

A summary of groundwater conditions during the monitoring program is provided in **Table 7**.

Table 7: Summary of Groundwater Observations

Borehole / Monitoring Well ID	Static Groundwater Measurements (mBGS)	
	Upon Completion of Drilling on January 2, 2026	Measured on January 30, 2026
BH26-01 / MW26-01	Dry	1.5

4.8 Corrosivity Analysis

One (1) sample was analyzed for chloride, sulphate, and sulphide concentrations, pH, electrical conductivity/resistivity, and redox potential at the Site. Laboratory data were compared to the ANSI/AWWA corrosivity rating system (provided in **Appendix C**) to determine the corrosive nature of the tested materials. A sample scoring greater than 10 points is considered to represent a corrosive environment with respect to grey or cast-iron alloys; other considerations, including the use of de-icing salts or stray electrical currents, to name a few, have not been considered. Additional analysis or testing may be required for alternative material types (i.e., copper, aluminum, etc.). **Table 8** summarizes the results for the Site and the total allotted points based on the rating system.

Table 8: Corrosivity Analytical Result and ANSI/AWWA Point Rating Summary

Sample	Parameter	Redox Potential (mV)	Sulphides (%)	Moisture Content (%)	pH	Resistivity (ohms-cm)	Total Points
BH26-01/ SS4	Value	254	<0.01	19	8.52	907	**14**
	Rating	0	2	2	0	10	

Based on the test results, corrosion conditions at the Site do appear to be significant. It is noted that there may be overriding factors in assessments of corrosion potential, such as the application and leaching of de-icing salts and stray electrical currents, to name a few. It is recommended that appropriate galvanization be selected, or adequate sacrificial thickness be provided to accommodate the anticipated corrosive nature of the material over the design life of the Site.

Additionally, PRI recommends that the structural engineer consider corrosivity potential for the Site based on the final design and provide considerations for buried utilities, steel piles and reinforcement rebar, as needed.

Laboratory test results for water-soluble sulphate concentration were 48 micrograms per gram (µg/g) and were compared to Table 3 of CSA A23.1-09 to assess the risk of sulphate attack on cementitious materials. Based on this, sulphate attack does not appear to be significant, and Type GU cement should be appropriate for most structural components used in concrete mix designs, with final design considerations to be determined by the structural engineer.

5 Geotechnical Recommendations

The following recommendations are intended for the design and construction of the proposed elevator. Recommendations are based on the borehole information described in **Section 4**. While PRI believes our findings are representative, conditions may vary beyond the locations investigated. If significant differences in the subsurface conditions described above are found later, particularly during construction or as more information becomes available, PRI should be contacted immediately to revise our findings and recommendations, as necessary.

Recommendations are intended for Designers and are not intended as instructions to Contractors, who should perform their own investigations to confirm any conditions that may affect construction schedules, costs and selected methodologies. Recommendations in this report must not be used by third parties without the express written consent of PRI.

5.1 Summary of Structural Information and Assumptions

The following summarizes PRI's understanding of the proposed works, based on information provided by the Durham District School Board (DDSB):

- A proposed elevator structure to be constructed external to the existing school building footprint, in the vicinity of borehole BH26-01;
- Associated localized interior renovations connecting the proposed elevator to the existing structure;

No information regarding the type, depth, condition, or configuration of the existing building foundations was provided or reviewed as part of this investigation. The existing foundations were not explored, exposed, or assessed. The subsurface investigation was completed to characterize subsurface conditions in the area of the proposed elevator construction only and does not constitute an evaluation of the existing building foundations or substructure.

Based on the subsurface conditions encountered at the borehole location, including the presence of fill and very soft to firm fine-grained soils, very dense native materials at depth, it is anticipated that shallow foundations will not be suitable for the proposed elevator structure, and deep foundations are expected to be required, subject to detailed design.

Final foundation type selection (e.g., driven piles, micropiles, or other deep foundation systems), foundation geometry, and design parameters will depend on final structural loading information, constructability considerations, and contractor means and methods, none of which were available at the time of this report.

5.2 General

The following recommendations are based on the subsurface conditions encountered at the borehole location, the scope of work completed to date, and the preliminary understanding of the proposed elevator construction. Final foundation selection, geometry, and design parameters shall be confirmed during detailed design once structural loading information, foundation layout, and construction details are available.

5.2.1 Site Preparation

Prior to grading and earthworks operations, all organic materials and other deleterious materials shall be stripped from beneath proposed structures, grading areas, walkways, and access roads. Existing buried structural components and utilities within the construction footprint shall be decommissioned and removed, as required, prior to excavation to the proposed subgrade elevations.

Subgrade preparation shall be completed to the satisfaction of the Geotechnical Engineer. Where shallow excavation areas are present, proof-rolling may be carried out using a static sheepsfoot roller to identify soft or unstable subgrade conditions. Proof-rolling, when undertaken, shall be completed in the presence of qualified inspection personnel working under the direction of the Geotechnical Engineer.

Any loose or soft subsoils identified during excavation or subgrade preparation shall be removed and replaced with approved engineered fill placed and compacted in accordance with Section 5.4. Where excessive rutting, localized soft areas, or unexpected organic materials are encountered, the use of a geotextile separator (e.g., Terrafix 360R or approved equivalent) may be considered to limit subexcavation depths, subject to approval by the Geotechnical Engineer.

5.2.2 Excavations

Excavations should be constructed in accordance with the most recent version of the Occupational Health and Safety Act (OHSA). The existing overburden materials above the groundwater table can be classified as Type 3 material in accordance with O.Reg. 213/91 s.226 under OHSA. Thus, temporary excavation side-slopes within the soils should be sloped at 1 Horizontal to 1 Vertical (1H:1V), or they must be properly supported (shored). Localized zones of very soft clayey silt may behave as Type 4 soils and should be treated accordingly if instability is observed. Soils below the groundwater table (if encountered) are considered Type 4 and should be sloped at a minimum grade of 3H:1V from the excavation bottom.

A layer of topsoil/organics approximately 1.1 m thick was encountered beneath the surficial fill materials. This material is expected to contain organic matter and is not suitable for structural support. Excavations extending through this layer may exhibit localized instability, particularly under wet conditions. Side slopes within this material should be carefully monitored and flattened or supported as required to maintain safe working conditions.

Excavations should be protected from exposure to precipitation and associated ground surface runoff and should be inspected regularly for signs of instability. If localized instability is noted during excavation, or if wet conditions are encountered, side slopes should be flattened or supported, as required by regulations, to maintain safe working conditions. All excavations should comply with applicable local, state and federal safety regulations, including the current OHSA Excavation and Trench Safety Standards.

Temporary shoring may be needed when excavating close to utilities, property boundaries, and any other existing structures or site elements to prevent materials from sloughing and undermining these features. Shoring systems must be designed by a qualified Professional Engineer in the Province of Ontario. The Contractor shall be responsible for maintaining stable excavations for the project.

5.2.3 Groundwater Control

Groundwater was measured at 1.5 mBGS in the monitoring well. Based on the expected depths of excavation for the proposed development, groundwater may be encountered during construction, and localized seepage is anticipated. If encountered, groundwater is expected to be manageable using conventional measures such as sumps and filtered pumps within excavations.

A Permit to Take Water (PTTW) or Environmental Activity and Sector Registry (EASR) is not expected to be required as per the requirements of the Ministry of the Environment, Conservation and Parks. Based on groundwater conditions, there may be other overriding factors to consider (i.e., biological).

Where deep foundation elements (e.g., micropiles) extend below the groundwater table, temporary groundwater control measures may be required, and permitting requirements should be reviewed once final foundation depths, installation methods, and anticipated pumping rates are confirmed.

Water levels should be verified at the time of construction, and PRI should be contacted to review all final designs, anticipated construction timing, dewatering methods, and permitting requirements once final construction and design details are available

5.2.4 Material Reuse, Backfill and Compaction

Fill materials containing deleterious material (e.g. topsoil, rootlets, etc.) are not considered suitable for reuse as backfill or for supporting foundations, nor should they be used for any of the pavement base or sub-base materials.

If consideration is given to the reuse of excavated soils at the time of construction, it is recommended that all materials designated for reuse be inspected by the Geotechnical Engineer prior to and/or during construction, to confirm that no deleterious material is present. Cobbles and boulders content within reused material should be less than 5 % by mass. If cobble and boulder content exceeds this limit, the material should be screened to remove all material greater than 60 mm, or an approved equivalent must be used.

Prior to placing any fill, all subgrade surfaces must be approved by the Geotechnical Engineer as noted in **Section 5.2.1**. Materials used for fill should be placed in maximum 200 mm loose lifts and compacted to 100 % of the Standard Proctor Maximum Dry Density (SPMDD) below foundations and structural components, 98 % of the SPMDD beneath access roads, and 95 % of the SPMDD in general fill areas. Compaction operations should be completed using a self-propelled vibratory compactor or jumping-jack plate tamper where access is limited. Backfill loose lift thicknesses may need to be reduced to achieve the above noted compaction values based on compaction equipment utilized (i.e. small tampers or jumping-jack).

It is recommended that foundation backfill consist of free-draining, non-frost susceptible granular fill material, such as Ontario Provincial Standard and Specifications (OPSS) 1010 Granular ‘B’ Type I materials or approved equivalent.

Pipe bedding and service trench backfill shall be placed in accordance with applicable OPSD standards and manufacturer requirements. Imported granular materials or approved native soils may be used, subject to confirmation of suitability.

5.2.5 Frost Considerations

Based on OPSD 3090.101, the frost penetration depth for the site area is 1.2 m below the final exterior grades. High-density Styrofoam insulation, or an approved equivalent, should be considered to provide equivalent frost protection where sufficient soil cover does not exist for foundation elements or adequate resistance to frost heave is not anticipated.

5.3 Seismic Site Class

Shear wave velocity testing was completed by Frontwave Geophysics Inc. on February 15, 2026, using the Multi-Channel Analysis of Surface Waves (MASW) method to determine the average shear wave velocity within the upper 30 metres (V_S30) for seismic site classification purposes. The complete report is included as **Appendix D.**

The MASW survey consisted of both active and passive data acquisition along a 69 m survey line located adjacent to the proposed elevator addition area. The interpreted shear wave velocity profile indicates increasing shear wave velocity with depth. Seismic refraction analysis performed as part of the investigation indicated that the depth to bedrock at the site was beyond the effective investigation depth of the refraction method (estimated at approximately 23 m below ground surface).

The calculated V_S30 values ranged from 356 m/s to 421 m/s, with an average value of 386 m/s. Based on the average V_S30, the site corresponds to Seismic Site Class C. Given that the reported range approaches the Site Class C/D boundary, the Structural Engineer may elect to adopt Site Class D for conservatism.

5.4 Preliminary Foundation Design

Final details related to the design and construction of the proposed elevator were not available at the time of this report. The subsurface investigation was completed to characterize subsurface conditions in the vicinity of the proposed elevator only, and does not constitute an assessment of existing building foundations.

Based on the subsurface conditions encountered in BH26-01, including fill/topsoil overlying compact sand, underlain by a thick deposit of very soft to firm clayey silt with localized loose silty sand/silt and sand seams to depth, shallow foundations are not recommended for support of the proposed elevator structure due to settlement and constructability concerns. Deep foundations extending to a competent bearing stratum are anticipated to be required.

At this stage, foundation recommendations are preliminary. Based on the structural engineer's input indicating that driven pile installation is not feasible within the existing school environment due to vibration, noise, and constructability constraints, helical piles or micropiles are considered the most viable deep foundation options for the proposed elevator. Final foundation selection shall be based on confirmed structural loading requirements, constructability, access constraints, and contractor means and methods.

It is understood that the preliminary structural design does not include uplift (tension) loading or lateral load demand on the deep foundation elements, and that deep foundations are intended to support axial compression loads only, with lateral loads resisted by the elevator shaft walls and superstructure.

For preliminary design purposes, competent native soils encountered below approximately 16.0 mBGS and approaching DCPT refusal at approximately 18.2 mBGS are considered the preferred bearing and/or bond stratum for deep foundation load transfer. Deep foundations should not rely on fill or soft surficial soils for axial resistance. Given the anticipated embedment depths (potentially approaching 18 mBGS), micropiles are considered the more practical and predictable deep foundation option for the constrained school environment. Helical piles may be feasible; however, feasibility and achievable installation torque at depth shall be confirmed by the specialty contractor, and proof testing is recommended.

5.4.1 Micropiles – Preliminary Geotechnical Considerations

If micropiles are considered, the following should be addressed:

- Loading assumption: Micropiles are assumed to be subject to axial compression loading only, based on information provided by the structural engineer at the time of reporting.
- Load transfer mechanism: Axial compression resistance is expected to be developed primarily through grout-to-soil bond along the bonded length and, where applicable, end bearing, subject to final pile configuration, embedment depth, and installation method.
- Preliminary grout-to-soil bond strength (competent native soils only): Based on typical nominal grout-to-soil bond strengths provided in FHWA-SA-97-070, Micropile Design and Construction Guidelines, preliminary design may consider a bond value consistent with Type B (pressure-grouted) micropiles founded within competent native soils (below approximately 16.4 mBGS). A preliminary bond stress of 100 to 150 kPa may be used for design, subject to confirmation of the selected installation method and verification load testing.
- Bond length: The micropile bonded length shall be founded entirely within competent native soils at depth and shall not rely on fill or soft surficial materials for axial resistance.

- Verification: At least one verification load test in compression is recommended to confirm achievable axial capacity and load–displacement behaviour for the selected installation method, as required by the foundation designer.

The micropile bonded length should be selected such that the bond zone is entirely within the dense to very dense native stratum encountered below approximately 16.4 mBGS. With DCPT refusal encountered at approximately 18.2 mBGS, the available thickness of competent native soils for bond development may be limited to approximately 2 m below 16.4 mBGS. Preliminary calculations indicate this may be feasible for the anticipated axial loads using the assumed bond stresses; however, final bonded length, bond diameter, and design bond stress shall be confirmed by the micropile designer and verified by load testing.

If uplift (tension) loads are introduced during detailed design, additional geotechnical review, design refinement, and supplemental verification testing may be required.

5.4.2 Helical Piles – Preliminary Geotechnical Considerations

If helical piles are considered for support of the proposed elevator structure, the following geotechnical considerations should be addressed during detailed design and construction planning:

- Loading assumption: Helical piles are assumed to be subject to axial compression loading only, based on information provided by the structural engineer.
- Bearing stratum: Helical pile helices should be advanced into competent native soils at depth, such as the dense to very dense native silty sand encountered below the surficial very soft to soft fine-grained soils (below approximately 16.4 mBGS). Helical piles shall not rely on fill or soft surficial soils for axial resistance.
- Axial capacity and load transfer: Axial compression resistance is expected to be developed primarily through end bearing on the helix plates, subject to final helix configuration, embedment depth, and soil conditions encountered at the time of installation.
- Installation torque correlation (design-level parameter): Installation torque monitoring shall be used as the primary means of capacity verification. The helical pile designer shall establish a torque–capacity correlation factor (Kt) based on the selected pile configuration, shaft size, and installation method. A minimum acceptance torque shall be specified to demonstrate achievement of the required axial compression resistance. Installation torque shall be continuously monitored and recorded during installation.
- Constructability: Helical pile installation is anticipated to be suitable for the constrained site conditions and proximity to existing structures, as installation generally produces minimal vibration and noise compared to driven pile systems.

- Verification: Proof or verification load testing in compression is recommended, as required by the foundation designer, to confirm achievable axial capacity, validate the torque capacity correlation, and assess load–displacement behaviour under serviceability loading.

With DCPT refusal encountered at approximately 18.2 mBGS, the available thickness of competent native soils for helix bearing below approximately 16.4 mBGS may be limited to approximately 2 m. Helical pile design shall account for this limited embedment zone, including selection of helix configuration, pile shaft capacity, and termination criteria to accommodate elevated installation torque or refusal that may be encountered in very dense native soils and near bedrock, while still achieving the required axial compression resistance. Helical piles should not be designed to bear directly on bedrock unless specifically intended by the foundation designer. Final design parameters and acceptance criteria shall be confirmed by the helical pile designer and verified by installation records and load testing.

5.5 Slab-On-Grade (Non-structural)

Where interior slab-on-grade construction is proposed for non-structural floor areas, slabs shall be supported on approved engineered fill placed and compacted in accordance with Section. 5.2.4. Subgrade preparation shall be reviewed by the Geotechnical Engineer prior to concrete placement.

Slabs should be reinforced to control cracking associated with shrinkage and temperature effects. Perimeter and slab drainage shall be provided where required to control groundwater and infiltration.

5.6 Soil Retaining Structure

Lateral earth pressures acting on buried walls, foundation walls, elevator pit walls, or other soil-retaining structures may be estimated using conventional earth pressure theory, based on the anticipated soil conditions, wall movement characteristics, and groundwater conditions at the time of construction. In general, lateral earth pressures may be expressed as:

$$P = K(\gamma h+q)$$

Where:

- P = lateral earth pressure acting at depth ‘h’ in kPa
- K = earth pressure coefficient (at-rest, active, or passive, as applicable)
- γ = bulk unit weight
- h = depth to point of interest in metres
- q = equivalent value of surcharge on the ground surface in kPa

Selection of the appropriate earth pressure coefficient (K_o, K_a, or K_p) should be based on the anticipated wall movement and restraint conditions, in accordance with accepted geotechnical practice and applicable design standards. Typical values for granular and cohesive soils may be adopted unless more refined analyses are completed as part of detailed design.

Where groundwater is present or may develop behind buried structures, hydrostatic water pressures shall be considered separately and added to the soil pressures, unless positive drainage measures are provided and demonstrated to be effective. For soil located below the groundwater level, the submerged unit weight of the soil should be used in calculating effective stresses.

This section applies to soil-retaining structural elements only and does not imply lateral load resistance by deep foundation elements, which are assumed to support axial loads only unless noted otherwise by the structural engineer.

Final earth pressure parameters and design pressures should be confirmed by the structural engineer during detailed design, taking into account final grades, backfill materials, drainage provisions, and construction sequencing.

6 Construction Supervision and Limitations

The data, conclusions and recommendations which are presented in this geotechnical report, and the quality thereof, are based on a scope of work authorized by the Client. While we believe the borehole information to be representative of Site conditions in the investigated areas, subsurface conditions between and beyond sampled locations may vary. If significant differences in any of the subsurface conditions described in this report are found, PRI should be contacted immediately to revise our findings and recommendations, if necessary.

Our comments on construction considerations are provided, but are not intended as instructions to Contractors, nor shall they be interpreted as specifications for construction. Contractors bidding shall make their own interpretations of factual information to determine how subsurface conditions may affect their methods, costs and schedules.

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

We trust this meets your current requirements. Please do not hesitate to contact the undersigned if you have any questions.

Yours truly,

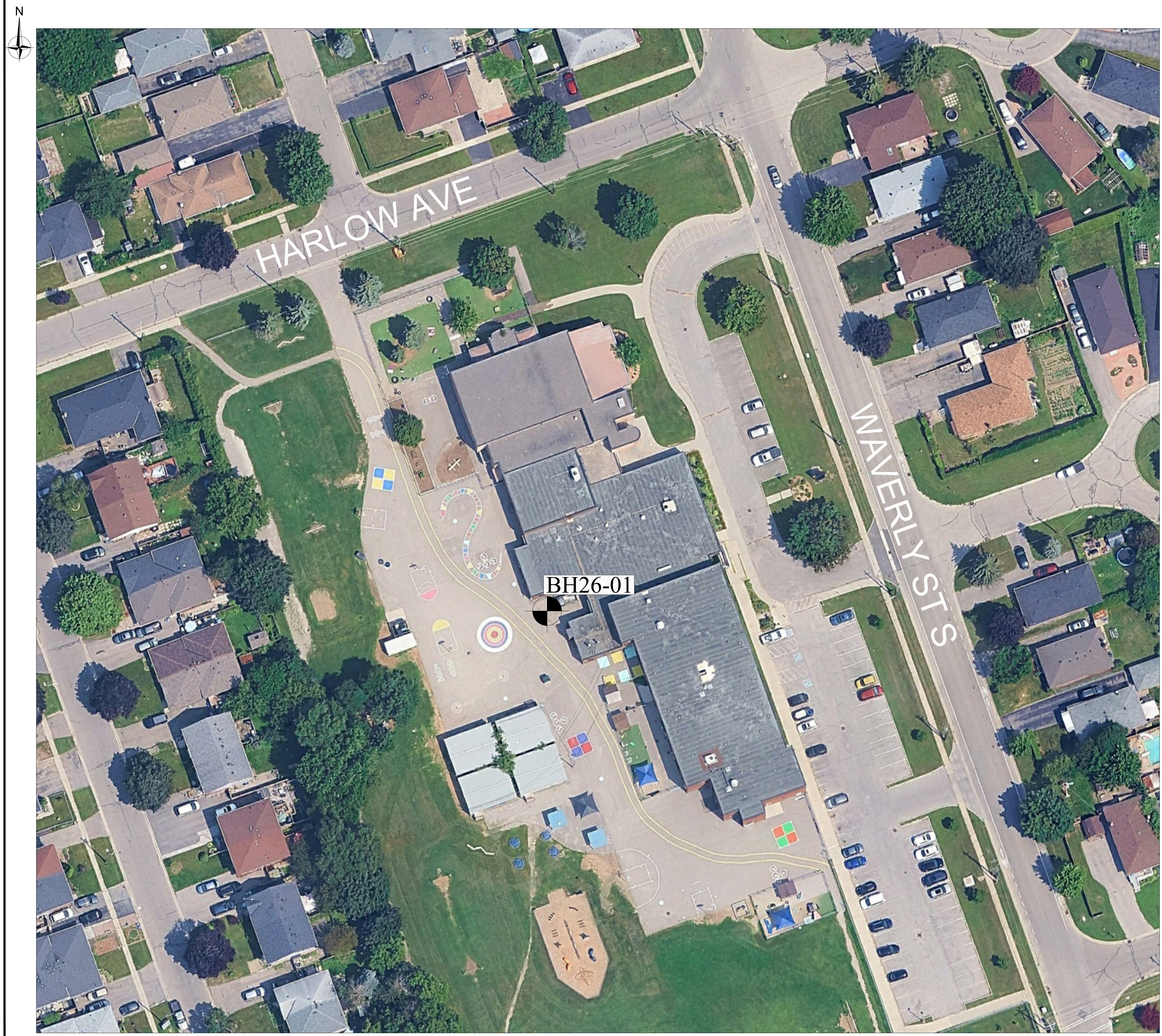
PRI Engineering Corp.

VGledhill

Esraa Shamess, E.I.T
Project Lead

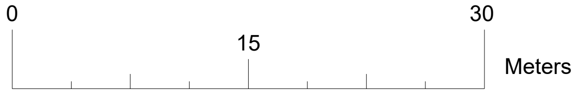
Vikki Gledhill, P. Eng.
Director, Engineering Services

Figures



KEY MAP

APPROXIMATE BOREHOLE ID AND LOCATION		
ID	**LATITUDE**	**LONGITUDE**
BH26-01	43.888574	-78.887597



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DDSB
Ignite Learning

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WHITBY ON L1R 2K6 CANADA

NOTES:
1. KEY MAP FROM GOOGLE MAPS AND USED FOR REFERENCE PURPOSES ONLY.
2. APPROXIMATE BOREHOLE, LOCATIONS ACQUIRED USING A HAND HELD GPS UNIT.
3. CONTRACTOR TO VERIFY LOCATIONS OF ANY UNDERGROUND UTILITIES PRIOR TO ANY GROUND DISTURBANCE.

LEGEND

BH26-## ◕	APPROXIMATE BOREHOLE ID AND LOCATION

00	ISSUED FOR REPORT	30JAN26
REV NO.	**ISSUANCE**	**DATE**

PROJECT NAME:

WAVERLY PUBLIC
SCHOOL 100
WAVERLY ST,
OSHAWA, ONTARIO

DRAWING NAME:

BOREHOLE
LOCATION PLAN

PROJ. NO.:	**DWG. BY:**	**CHKD. BY:**	**APPR. BY:**
25-578	DRK	ES	VG

DRAWING NUMBER: FIGURE 1

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Appendix A

Borehole Explanation Form, Borehole Logs

BOREHOLE LOG EXPLANATION FORM

This explanatory section provides the background to assist in the use of the borehole logs. Each of the headings used on the borehole log, is briefly explained.

DEPTH

This column gives the depth of interpreted geologic contacts in metres below ground surface.

STRATIGRAPHIC DESCRIPTION

This column gives a description of the soil based on a tactile examination of the samples and/or laboratory test results. Each stratum is described according to the following classification and terminology.

Soil Classification*		Terminology	Proportion
Silt & Clay	< 0.075 mm	"trace" (e.g. trace sand)	<10%
Sand	0.075 to 4.75 mm	"some" (e.g. some sand)	10% - 20%
Gravel	4.75 to 75 mm	adjective (e.g. sandy)	20% - 35%
Cobbles	75 to 300 mm	"and" (e.g. and sand)	35% - 50%
Boulders	>300 mm	noun (e.g. sand)	>50%

* Extension of USCS Classification system unless otherwise noted.

The use of the geologic term "till" implies that both disseminated coarser grained (sand, gravel, cobbles or boulders) particles and finer grained (silt and clay) particles may occur within the described matrix.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

COHESIONLESS SOIL		COHESIVE SOIL	
Compactness	Standard Penetration Resistance "N", Blows / 0.3 m	Consistency	Standard Penetration Resistance "N", Blows / 0.3 m
Very Loose	0 to 4	Very Soft	0 to 2
Loose	4 to 10	Soft	2 to 4
Compact	10 to 30	Firm	4 to 8
Dense	30 to 50	Stiff	8 to 15
Very Dense	Over 50	Very Stiff	15 to 30
		Hard	Over 30

The moisture conditions of cohesionless and cohesive soils are defined as follows.

COHESIONLESS SOILS	COHESIVE SOILS		
Dry	DTPL	-	Drier Than Plastic Limit
Moist	APL	-	About Plastic Limit
Wet	WTPL	-	Wetter Than Plastic Limit
Saturated	MWTPL	-	Much Wetter Than Plastic Limit

STRATIGRAPHY

Symbols may be used to pictorially identify the interpreted stratigraphy of the soil and rock strata.

MONITOR DETAILS

This column shows the position and designation of standpipe and/or piezometer ground water monitors installed in the borehole. Also the water level may be shown for the date indicated.

- ● Standpipe
- ▲ Piezometer
- ☰ Screened Interval
- ▧ Borehole Seal (Peltonite, Bentonite or Hole Plug)
- ■ Geotextile Material / Liner
- ▧ Borehole Seal (Bentonite Grout)
- ▨ Cement Seal
- ▦ Granular Backfill
- ▩ Granular (Filter) Pack
- □ Native Soil Backfill / Cave / Slough

Where monitors are placed in separate boreholes, these are shown individually in the "Monitor Details" column. Otherwise, monitors are in the same borehole. For further data regarding seals, screens, etc., the reader is referred to the summary of monitor details table.

SAMPLE

These columns describe the sample type and number, the "N" value, the water content, the percentage recovery, and Rock Quality Designation (RQD), of each sample obtained from the borehole where applicable. The information is recorded at the approximate depth at which the sample was obtained. The legend for sample type is explained below.

SS = Split Spoon
ST = Thin Walled Shelby Tube
AS = Auger Flight Sample
CC = Continuous Core
GS = Grab Sample
CS = Channel Sample
WS = Wash Sample
RC = Rock Core

$$\% \text{ Recovery} = \frac{\underline{\text{Length of Core Recovered Per Run}}}{\text{Total Length of Run}} \text{ x } 100$$

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD (%)
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

TEST DATA

The central section of the log provides graphs which are used to plot selected field and laboratory test results at the depth at which they were carried out. The plotting scales are shown at the head of the column.

Dynamic Penetration Resistance - The number of blows required to advance a 51 mm diameter, 60° steel cone fitted to the end of 45 mm OD drill rods, 0.3 m into the subsoil. The cone is driven with a 63.5 kg hammer over a fall of 750 mm.

Standard Penetration Resistance - Standard Penetration Test (SPT) "N" Value - The number of blows required to advance a 51 mm diameter standard split-spoon sampler 300 mm into the subsoil, driven by means of a 63.5 kg hammer falling freely a distance of 750 mm. In cases where the split spoon does not penetrate 300 mm, the number of blows over the distance of actual penetration in millimetres is shown as $\frac{xBlows}{mm}$

Water Content - The ratio of the mass of water to the mass of oven-dry solids in the soil expressed as a percentage.

W_P - Plastic Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

W_L - Liquid Limit of a fine-grained soil expressed as a percentage as determined from the Atterberg Limit Test.

REMARKS

The last column describes pertinent drilling details, field observations and/or provides an indication of other field or laboratory tests that were performed.

BORING NUMBER BH25-01

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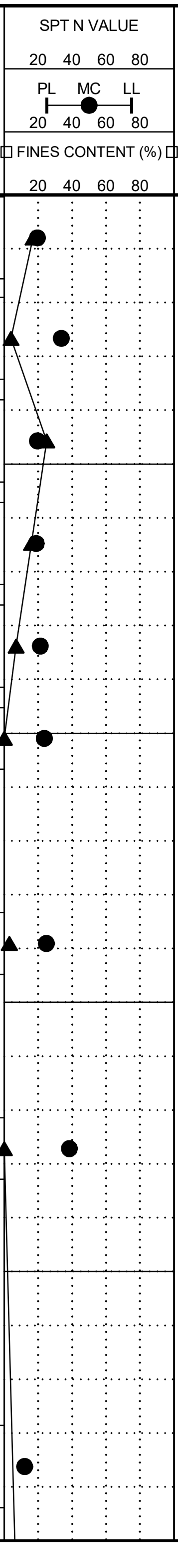
CLIENT Durham District School Board
PROJECT NAME Waverly Public School
PROJECT NUMBER 25-578
PROJECT LOCATION Oshawa, ON
DATE STARTED 26-1-2 **COMPLETED** 26-1-2
GROUND ELEVATION Not Determined
DRILLING CONTRACTOR TCI Field Services
GROUND WATER LEVELS:
DRILLING METHOD 152mm O.D. Solid Stem Auger
LOGGED BY VG/MP **CHECKED BY** VG
AT END OF DRILLING ---
NOTES Lat: 43.888574, Long: -78.887597
AFTER DRILLING 1.5 m

DEPTH (m)	ELEVATION (mASL)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (kPa)	MOISTURE CONTENT (%)	REMARKS AND TESTS
0.1 0.3 0.4		ASPHALT (100 mm): GRANULAR FILL: Brown sand and gravel GRANULAR FILL, some silt and clay, moist, compact	SS 1	75	11-7-10-7 (17)		20	Borehole was open and dry upon completion of drilling.
		FILL: Brown silty sand FILL, some gravel, moist, compact	SS 2	100	2-2-2-2 (4)		34	
1.5 2.0		TOPSOIL: SAND: Grey/brown SAND, trace silt, saturated, compact	SS 3	75	9-11-14-15 (25)		20	
			SS 4	92	5-7-9-11 (16)		19	Groundwater level measured at 1.5 m mBGS in monitoring well on January 30, 2026.
3.3		CLAYEY SILT: Grey CLAYEY SILT, trace sand, APL, very soft to firm	SS 5	100	3-4-3-3 (7)		21	
4.0		- WTPL	SS 6	94	1-0-0 (0)		24	
			VA 1					VA1: Intact: 74 kPa Remoulded: 11 kPa
		- APL	SS 7	100	1-1-2 (3)		25	GSA SS7: Gravel: 0% Sand: 1% Silt: 56% Clay: 43%
6.0			VA 2					ALT SS7: LL: 25% PL: 14% PI : 11
		- WTPL	SS 8	100	0-0-0 (0)		38	VA2: Intact: 65 kPa Remoulded: 16 kPa
8.0			VA 3					VA3: Intact: 21 kPa Remoulded: 7 kPa
10.0			SS 9	21	0		12	



(Continued Next Page)

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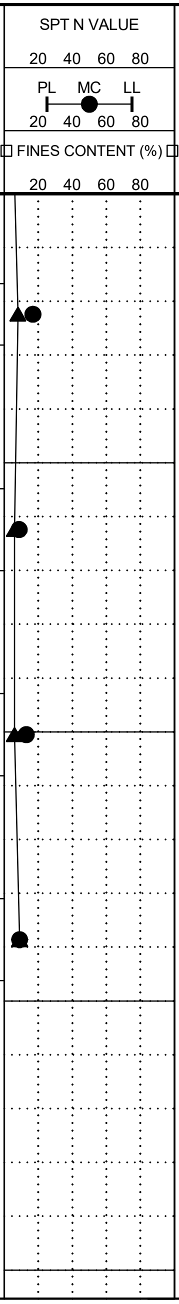
BORING NUMBER BH25-01

CLIENT Durham District School Board
PROJECT NAME Waverly Public School
PROJECT NUMBER 25-578
PROJECT LOCATION Oshawa, ON

DEPTH (m)	ELEVATION (mASL)	GRAPHIC LOG	MATERIAL DESCRIPTION	MONITOR WELL DETAILS	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (kPa)	MOISTURE CONTENT (%)	REMARKS AND TESTS
			CLAYEY SILT: Grey CLAYEY SILT, trace sand, APL, very soft to firm *(continued)*							
			- Trace gravel		SS 10	100	4-4-4 (8)		17	
10.9			SILTY SAND: Brown SILTY SAND, trace clay, saturated, loose							
12.0										
			- 125 mm gravel seam		SS 11	67	8-3-3-5 (6)		9	
13.7			CLAYEY SILT: Grey CLAYEY SILT, trace sand, APL, firm							
14.0					SS 12	63	4-2-4-5 (6)		13	
15.2			SILT AND SAND: Grey SILT AND SAND, some clay, trace gravel, moist, loose		SS 13	100	3-4-5-6 (9)		9	GSA SS13: Gravel: 7% Sand: 40% Silt: 40% Clay: 13%
15.8			Sampling terminated at 15.8 m below ground surface in SILT AND SAND. DCPT advanced from 16.4 to 18.2 mBGS.							
16.0							10 38 31 16 16 21 16 20 18 21 47 100/75 mm			
18.0										
18.2			Borehole terminated at 18.2 m below ground surface upon DCP refusal.							



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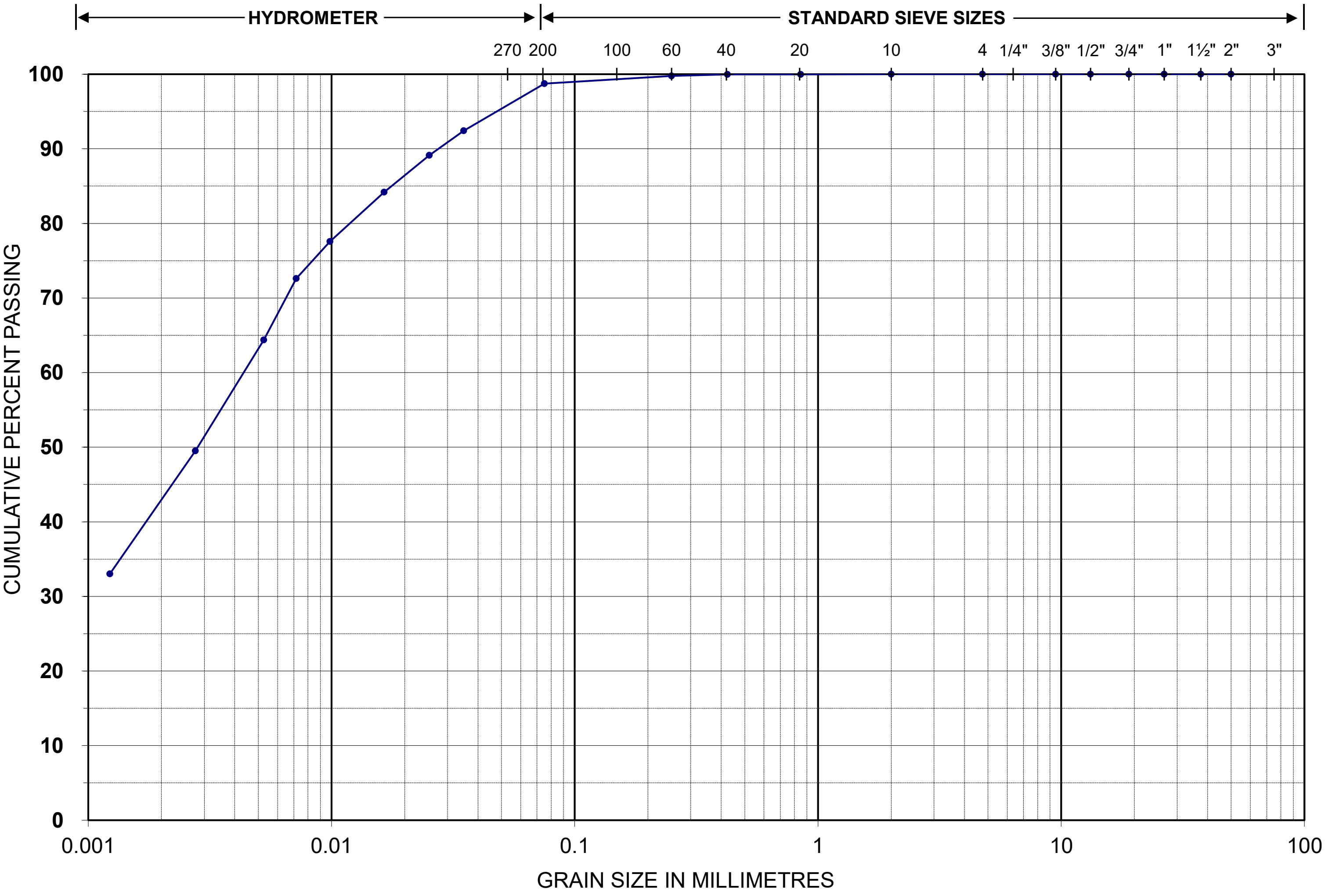
Appendix B

Geotechnical Laboratory Results

PARTICLE SIZE DISTRIBUTION
LS - 702

Project Name: Waverly Public School
Project No.: 25-578
Sample Date: 2-Jan-26
Borehole/Test Pit ID.: BH26-01
Sample No./Depth: SS7 / 5.3 - 5.8 m
LAB ID: 26HYD-009



Silt or Clay	Sand	Gravel

Sieve Size (mm)	% Passing
37.5	100.0
26.5	100.0
19.0	100.0
13.2	100.0
9.5	100.0
4.750	100.0
2.000	100.0
0.850	100.0
0.425	100.0
0.250	99.8
0.075	98.7

Hydrometer (mm)	% Passing
0.035	92.4
0.025	89.1
0.016	84.2
0.010	77.6
0.007	72.6
0.005	64.4
0.003	49.5
0.001	33.0

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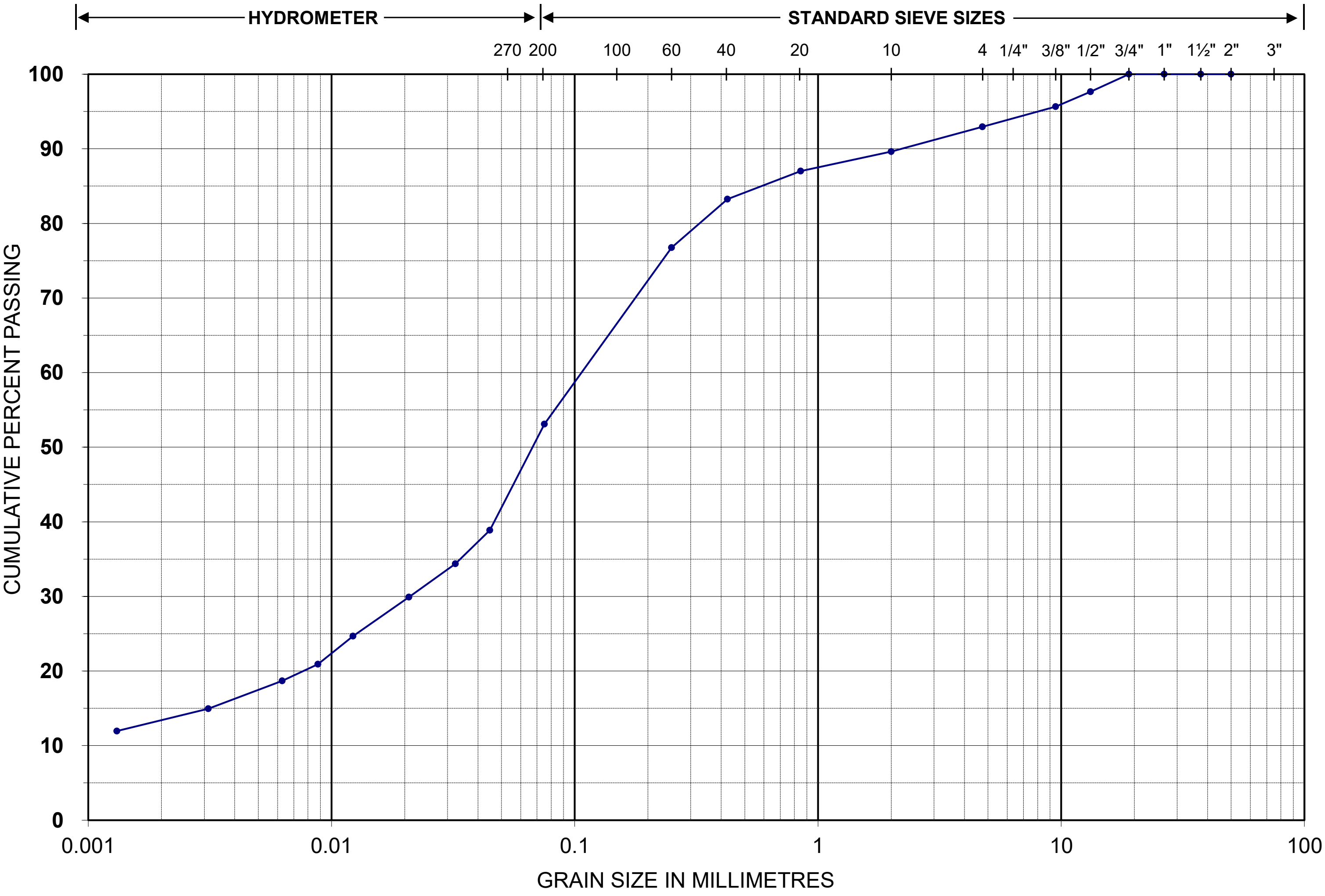
205 St.George Street, Unit 2, Lindsay, ON, K9V 5Z9
☎ (705) 702-3921
info@priengineering.com
www.priengineering.com

PARTICLE SIZE DISTRIBUTION
LS - 702

Project Name: Waverly Public School
Project No.: 25-578
Sample Date: 2-Jan-26
Borehole/Test Pit ID.: BH26-01
Sample No./Depth: SS13 / 15.2 - 15.8 m
LAB ID: 26HYD-010



Silt or Clay	Sand	Gravel

Sieve Size (mm)	% Passing
37.5	100.0
26.5	100.0
19.0	100.0
13.2	97.7
9.5	95.6
4.750	92.9
2.000	89.6
0.850	87.0
0.425	83.2
0.250	76.8
0.075	53.1

Hydrometer (mm)	% Passing
0.045	38.9
0.032	34.4
0.021	29.9
0.012	24.7
0.009	20.9
0.006	18.7
0.003	14.9
0.001	12.0

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ATTERBERG LIMITS
ASTM D4318

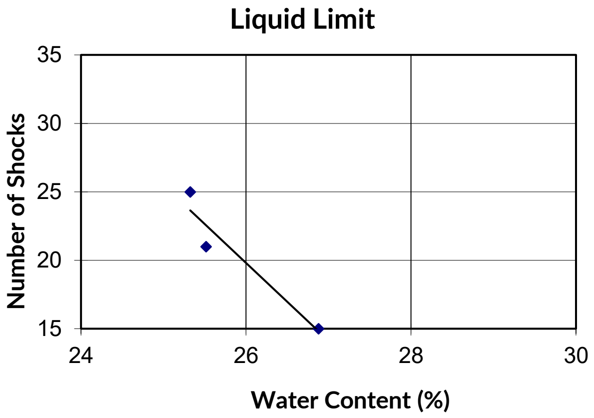
Project Name: Waverly Public School
Project No.: 25-578
Sample Date: 2-Jan-26
Borehole/Test Pit ID.: BH26-01
Sample No./Depth: SS7 / 5.3 - 5.8 m
LAB ID: 26ALT-005

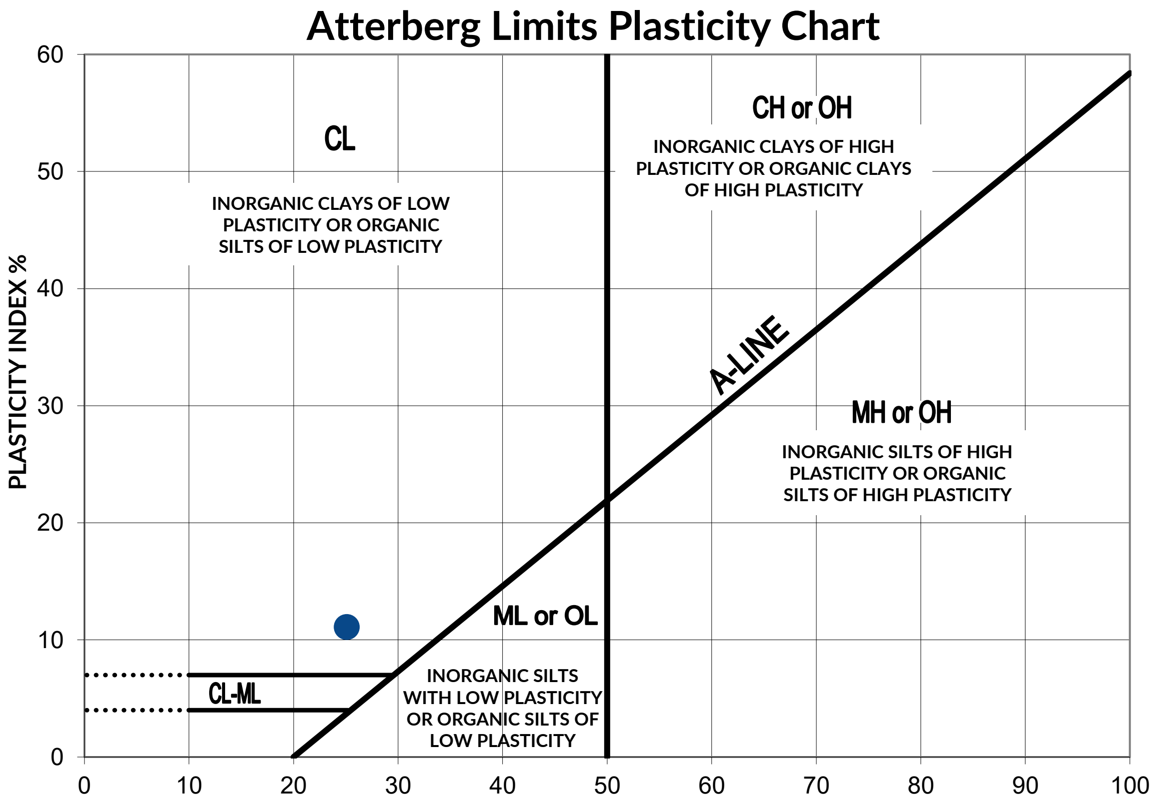
SAMPLE RESULTS

Liquid Limit, (W_L)	25
Plastic Limit, (W_P)	14
Plasticity Index (I_P=W_L-W_P)	11
Natural Water Content, W	24
Liquidity Index (I_L=W-W_P/W_L-W_P)	1

CONTROL RESULTS

Liquid Limit, (W_L)	34
Plastic Limit, (W_P)	18
Plasticity Index (I_P=W_L-W_P)	16

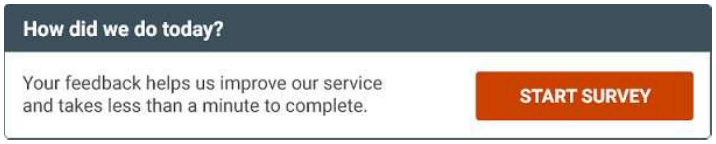




Approver: VG
Issue Date: 2022-05-30
Issue/Revision Number: Issue 1, Revision 1

Appendix C

Corrosivity Laboratory Results and ANSI/AWWA Rating System



How did we do today?

Your feedback helps us improve our service and takes less than a minute to complete.

FINAL REPORT

CA40018-JAN26 R1

25-578, Waverly P.S.

Prepared for

PRI Engineering Corp.

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	PRI Engineering Corp.	Project Specialist	Jill Campbell, B.Sc.,GISAS
Address	2161 Whittington Drive Cavan Monaghan, Ontario K9J 0G5. Canada	Laboratory	SGS Canada Inc.
		Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Vikki Gledhill		
Telephone	705-702-3921	Telephone	2165
Facsimile		Facsimile	705-652-6365
Email	vikki.gledhill@priengineering.com	Email	jill.campbell@sgs.com
Works #		SGS Reference	CA40018-JAN26
Project	25-578, Waverly P.S.	Received	2026-01-08
Reference		Approved	01/15/2026
Batch		Report Number	CA40018-JAN26 R1
Samples	SOIL (1)	Date Reported	01/15/2026

COMMENTS

Temperature of Sample upon Receipt: 18 degrees C

Cooling Agent Present:No

Custody Seal Present:No

Chain of Custody Number:044943

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Jill Campbell, B.Sc.,GISAS

Jill Campbell

TABLE OF CONTENTS

MATRIX: SOIL

Sample Number 5
Sample Name BH26-01/SS4
Sample Matrix Soil
Sample Date 2026-01-02 00:00

Parameter	Units	RL		Result
Corrosivity Index				
Corrosivity Index	none	1		15
pH	pH Units	0.05		8.52
Soil Redox Potential	mV	no		254
Sulphide (Na2CO3)	%	0.01		< 0.01
Resistivity (calculated)	ohms.cm	-9999		907
General Chemistry				
Conductivity	uS/cm	2		1100
Metals and Inorganics				
Sulphate	µg/g	0.4		48
Other (ORP)				
Chloride	µg/g	0.4		580

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0217-JAN26	µg/g	0.4	<0.4	NV	35	99	80	120	106	75	125
Sulphate	DIO0217-JAN26	µg/g	0.4	<0.4	14	35	99	80	120	98	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0051-JAN26	%	0.01	< 0.01								

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0196-JAN26	uS/cm	2	< 2	0	20	99	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0196-JAN26	pH Units	0.05	NA	0		101			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS	Insufficient sample for analysis.
RL	Reporting Limit.
↑	Reporting limit raised.
↓	Reporting limit lowered.
NA	The sample was not analysed for this analyte
ND	Non Detect

Results relate only to the sample tested.
Data reported represent the sample as submitted to SGS.

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Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions.htm (Printed copies are available upon request.)

Test method information available upon request.
"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS Canada Inc. statement of conformity decision rule does not consider uncertainty when analytical results are compared to a specified standard or regulation.

-- End of Analytical Report --

SGS

Request for Laboratory Services and CHAIN OF CUSTODY

Industries & Environment - Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

No: 044943

Page ______ of ______

Laboratory Information Section - Lab use only

Received By: Katie Larocque
Received Date: 01 / 08 26 (mm/dd/yy)
Received Time: 11 : 50 (hr : min)

Received By (signature):
Custody Seal Present: Yes ☐ No ☑
Custody Seal Intact: Yes ☐ No ☑

Cooling Agent Present: Yes ☐ No ☑ Type:______
Temperature Upon Receipt (°C) 18 , ____ , ____

LAB LIMS #: CA40018 Jan 25

REPORT INFORMATION

Company: PRI
Contact: vikki.gledhill@priengineering.com
Address: 2161 Whittington Dr.
Cavan-Monaghan
Phone:
Fax:
Email: Vikki Gledhill

INVOICE INFORMATION

☒ (same as Report Information)
Company:
Contact:
Address:
Phone:
Email:

Quotation #:
P.O. #:
Project #: 25-578
Site Location/ID: Waverly P.S.

TURNAROUND TIME (TAT) REQUIRED

☐ Client Regular TAT ☒ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day

RUSH TAT (Additional Charges May Apply): ☐ 1 Day ☐ 2 Days ☐ 3 Days ☐ 4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:

***NOTE:** DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

REGULATIONS

☐ O.Reg 153/04 ☐ O.Reg 406/19
☐ Table 1 ☐ Res/Park Soil Texture:
☐ Table 2 ☐ Ind/Com ☒ Coarse
☐ Table 3 ☐ Agri/Other ☐ Medium/Fine
☐ Table ______ Appx. ______
Soil Volume ☐ <350m3 ☐ >350m3

Other Regulations:
☐ Reg 347/558 (3 Day min TAT)
☐ PWQO ☐ MMER
☐ CCME ☐ Other:
☐ MISA
☐ ODWS Not Reportable *See note

Sewer By-Law:
☐ Sanitary
☐ Storm
Municipality:

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

ANALYSIS REQUESTED

	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF ~~BOTTLES~~ Bag	MATRIX	M & I				SVOC		PCB	PHC		VOC		Pest	Other (please specify)						SPLP	TCLP	COMMENTS:
						Field Filtered (Y/N)	Metals & Inorganics incl CrVI, CN,Hg pH,(B(HWS),EC,SAR-soil) (Cl, Na-water)	Full Metals Suite ICP metals plus B(HWS-soil only) Hg, CrVI	ICP Metals only Sb,As,Ba,Be,B,Cd, Cr,Co,Cu,Pb,Mo,Ni,Se,Ag,Tl,U,V,Zn	PAHs only	SVOCs all incl PAHs, ABNs, CPs	PCBs Total ☐ Aroclor ☐	F1-F4 + BTEX	F1-F4 only no BTEX	VOCs all incl BTEX	BTEX only	Pesticides Organochlorine or specify other	Corrosivity				Sewer Use: Specify pkg:	Water Characterization Pkg General ☐ Extended ☐	Specify tests ☐ Metals ☐ VOC ☐ 1,4-Dioxane ☐ OCP ☐ ABN	Specify tests ☐ M&I ☐ VOC ☐ PCB ☐ B(a)P ☐ ABN ☐ Ignit.	
1	BH26-01/SS4	01/02/06	-	1	S													X								
2																										
3																										
4																										
5																										
6																										
7																										
8																										
9																										
10																										
11																										
12																										

Observations/Comments/Special Instructions

Sampled By (NAME): Vikki Gledhill Signature: VGledhill Date: 01 / 05 / 26 (mm/dd/yy) Pink Copy - Client
Relinquished by (NAME): Sarah Beukeboom Signature: Sarah Beukeboom Date: 01 / 08 / 26 (mm/dd/yy) Yellow & White Copy - SGS

Revision #: 1.8
Date of Issue: 06 SEP 2024

Note: Submission of samples to SGS is acknowledgement that you have been provided direction on sample collection/handling and transportation of samples. {2} Submission of samples to SGS is considered authorization for completion of work. Signatures may appear on this form or be retained on file in the contract, or in an alternative format (e.g. shipping documents). {3} Results may be sent by email to an unlimited number of addresses for no additional cost. Fax is available upon request. This document is issued by the Company under its General Conditions of Service accessible at http://www.sgs.com/terms_and_conditions.htm. (Printed copies are available upon request.) Attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein.

Table A.1 Soil-test evaluation

Soil Characteristics Based on Samples Taken Down to Pipe Depth		
Resistivity—ohm-cm (based on water-saturated soil box):		Points[*]
	<1,500	10
	≥1,500–1,800	8
	>1,800–2,100	5
	>2,100–2,500	2
	>2,500–3,000	1
	>3,000	0
pH:		
	0–2	5
	2–4	3
	4–6.5	0
	6.5–7.5	0[†]
	7.5–8.5	0
	>8.5	3
Redox potential:		
	> +100 mV	0
	+50 to +100 mV	3.5
	0 to +50 mV	4
	Negative	5
Sulfides:		
	Positive	3.5
	Trace	2
	Negative	0
Moisture:		
	Poor drainage, continuously wet	2
	Fair drainage, generally moist	1
	Good drainage, generally dry	0

[*] Ten points or greater indicates that soil is corrosive to ductile-iron pipe; protection is needed. Refer to paragraph A.3 for a description of Uniquely Severe Environments and additional considerations.

[†] If sulfides are present and low (<100 mv) or negative redox-potential results are obtained, add three points for this range.

PRI ENGINEERING

Appendix D

Shear Wave Velocity Testing (MASW) Report

SHEAR WAVE VELOCITY TESTING
FOR SEISMIC SITE CLASSIFICATION
WAVERLY PUBLIC SCHOOL
100 WAVERLY STREET SOUTH, OSHAWA, ONTARIO

Submitted to:

PRI Engineering
A-2161 Whittington Drive
Cavan Monaghan, Ontario K9J 6X4

Attention:

Ms. Vikki Gledhill, P.Eng. Email: vikki.gledhill@priengineering.com

File No. F-26479

February 20, 2026

Frontwave Geophysics Inc.
Brampton, ON
(647) 514-4724
www.frontwave.ca

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1 INTRODUCTION

Frontwave Geophysics Inc. was retained by PRI Engineering to carry out a geophysical investigation for the proposed addition to Waverly Public School located at 100 Waverly Street South in Oshawa, Ontario.

The objective of the survey was to determine site designation for seismic site response based on average shear wave velocity value measured in the upper 30 m (V_S30). The multi-channel analysis of surface waves (MASW) method was used to obtain shear wave velocity profile.

The fieldwork was conducted on February 15, 2026. The location of the MASW survey line is shown in Figure 1.

This report describes the basic principles of MASW, survey design, interpretation method, and presents the results of the investigation in the chart and table format.

2 INVESTIGATION METHODOLOGY

Overview

The Multi-channel Analysis of Surface Waves (MASW) is a seismic method widely applied to produce shear wave velocity (V_S) profiles. It is based on the dispersive nature of Rayleigh or Love surface waves in layered media. Surface waves with longer wavelengths propagate deeper in the subsurface, hence, their phase velocity is more influenced by the elastic properties of deeper layers. The velocity of surface waves depends mainly on the shear wave velocity of the medium. The distribution of surface waves phase velocities as a function of wavelength (or frequency) can be visualized as a dispersion curve. The inverse problem is then solved by modelling the experimental data with a theoretical dispersion curve; the model parameters are typically limited to layer thickness and shear wave velocity with an assumption of horizontally layered strata. As a result of the inversion, a shear wave velocity depth profile is obtained. Figure 2 illustrates the overall procedure of the MASW method.

Two approaches different in data acquisition and processing can be implemented. The active method involves using artificial sources (e.g., sledgehammer, drop weight) to generate seismic energy, whereas the passive method utilizes energy generated by natural sources (wind, waves, microseismicity) and human activities (mostly vehicle traffic). The energy that can be generated with easily accessible active sources such as sledgehammers is typically concentrated within a relatively high frequency range, and the maximum depth of penetration for active surveys is limited to approximately 15-30 m, depending on the mass of the source and geology of the site. Ambient vibrations registered with the passive acquisition are usually of lower frequency and provide better resolution at greater depths. When survey logistics allow, the active and passive source methods are combined for obtaining well-resolved dispersion images over a wide frequency range, thus increasing the depth of investigation while retaining high resolution at shallow depths.

Legend

MASW Location of MASW survey line
(69 m geophone spread)

Image: Google Earth 2024

Date: 2026-02-20

File No: F-26479

FRONTWAVE GEOPHYSICS

Title: Survey location plan

Location: 100 Waverly St S
Oshawa, ON

Figure:
1

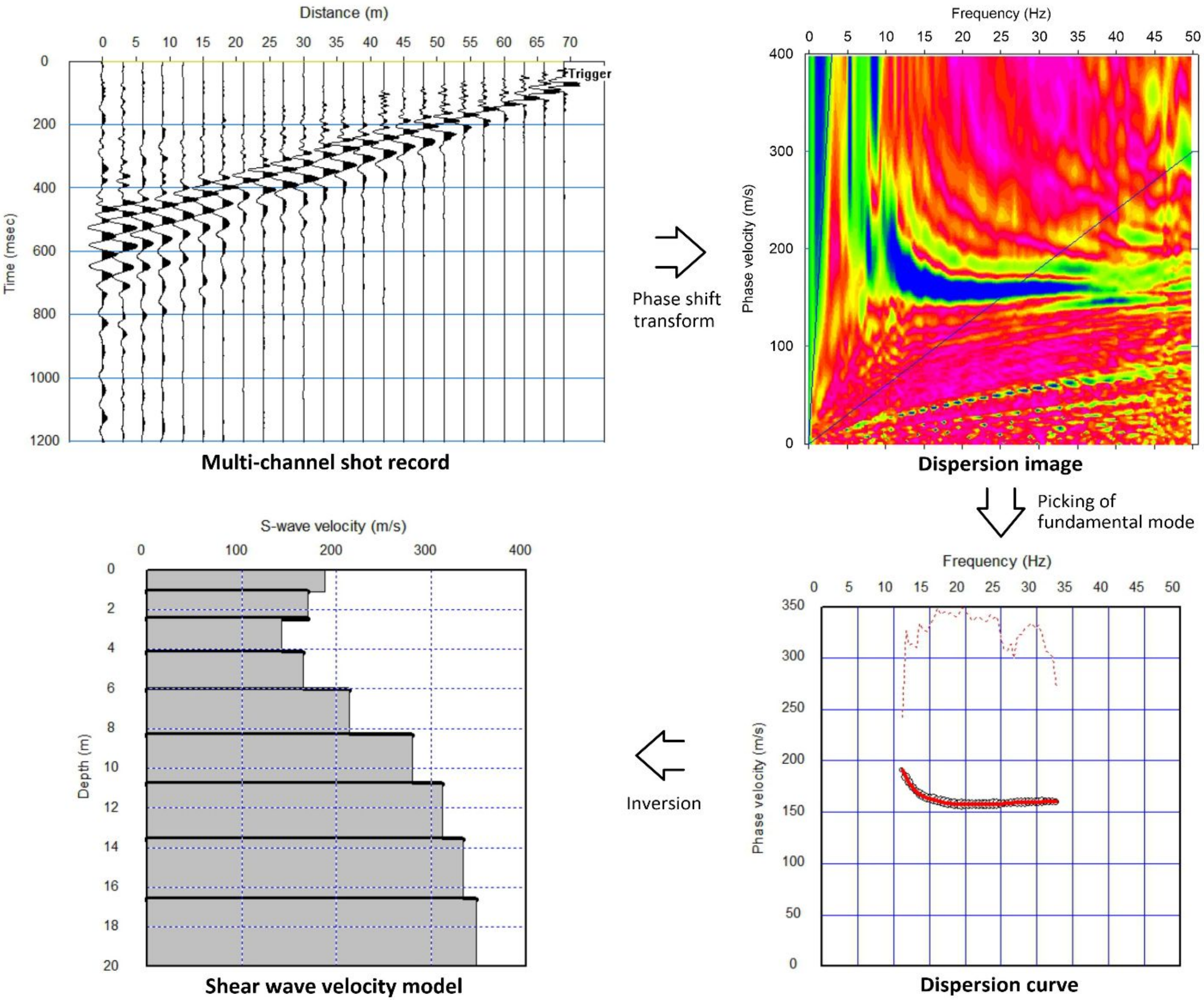


Figure 2 *The procedure of MASW data processing using the SeisImager SW software package.*

Survey Design

The acquisition layout consisted of 24 receivers in a linear array (spread), connected with a multicore cable to a DAQLink 4 seismograph. 4.5 Hz natural frequency vertical geophones were used for this survey. To optimize sampling of different wavelengths, two sets of measurements were conducted with spread lengths of 23 m and 69 m (1 m and 3 m spacing between geophones, respectively). Data collected with longer spreads provide a greater depth of investigation, whereas data collected with shorter geophone spacings ensure better resolution in the uppermost few meters of the subsurface.

An 8-kg sledgehammer was used as an energy source for active acquisition. Shots were executed at three to five locations per spread: two shots close to the ends of the spread and one to three shots within the spread. A total of 8 shot records was collected. The record length was set to 1500 ms with a 0.05 ms sampling interval.

For passive acquisition, a linear 24-channel array with 3 m spacing between geophones was used. Ambient wavefield was recorded for 10 minutes with a sampling interval of 2 ms.

Interpretation

A dispersion curve is obtained from each field record by converting the shot gather into a dispersion image and then identifying and picking the fundamental mode. A shear wave velocity profile is obtained through inversion of the dispersion curve by modelling the subsurface as a horizontally layered medium with the model parameters limited to the number of layers, their thickness and shear-wave velocity.

SeisImager SW software package was used for processing, picking and inversion of the MASW data.

Some variability among the dispersion curves and resulting models obtained from different shot records is always observed due to lateral velocity variations, near and far field effects, different signal-to-noise ratio, etc. Combining independent inversion results from multiple shot records improves the estimation of the actual shear wave velocity and provides an assessment of uncertainty. The results of the interpretation are presented in the form of the average shear wave velocity profile; the observed variability of the MASW data is reported as upper and lower bound velocity profiles.

Accuracy of the results

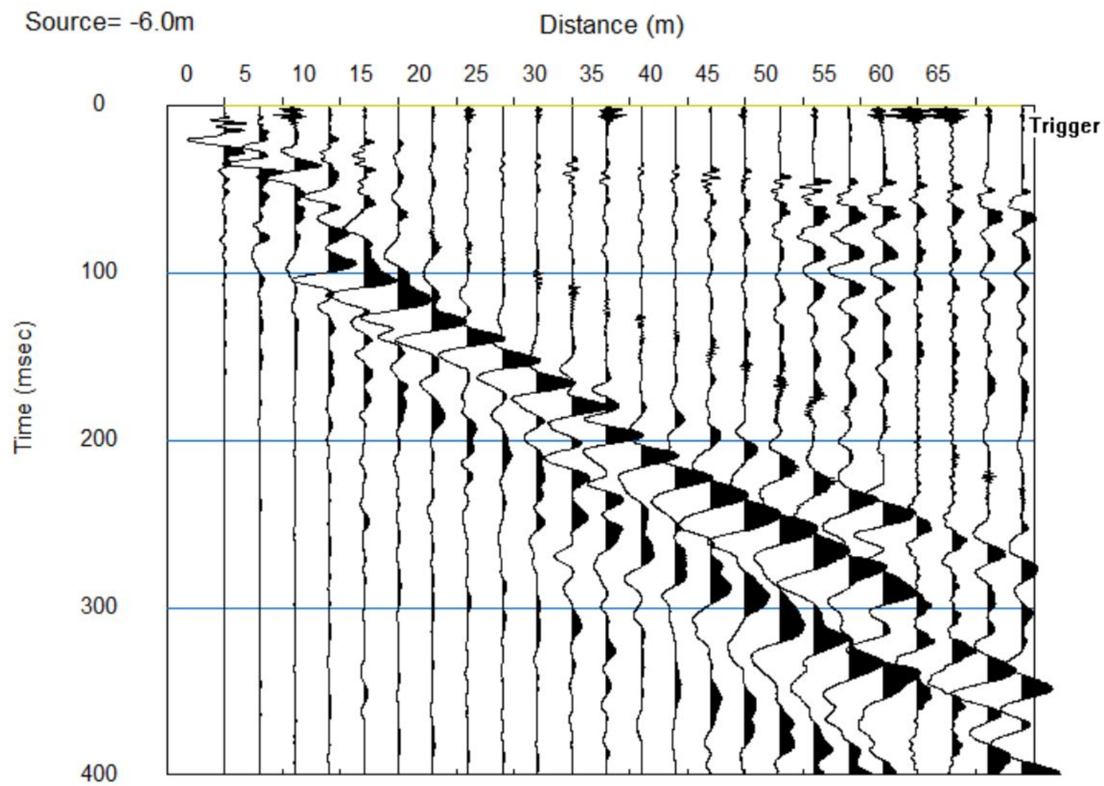
The accuracy of MASW generally depends on the complexity of the subsurface and specific site conditions (noise levels, topography, etc.). Lateral velocity variations and steeper bedrock topography increase the dispersion uncertainty. The presence of high velocity contrast layers such as bedrock will require the use of a-priory information to optimize model parameters for more accurate results. Hence, if the a-priory information is not available (e.g. when the data are overly noisy to carry out refraction analysis), the accuracy decreases.

The uncertainty of the resulting S-wave velocity depth profile is evaluated using the upper and lower bound velocity profiles. Typically, the error margin of average V_S30 value determined from MASW is within ±10%.

3 RESULTS

The collected surface wave data were of good quality; the dispersion images showed good resolution and covered a frequency range of approximately 6 to 60 Hz. Example shot record and MASW dispersion images obtained at this site are presented in Figure 3.

Seismic refraction analysis indicated that the depth to bedrock at this site was beyond the investigation depth of the refraction method (which was estimated to be approximately 23 m below the ground surface). Compressional (P) wave velocity measured in the overburden below the water table was approximately 1850 m/s.



Active acquisition (spread length 69 m)

Passive acquisition

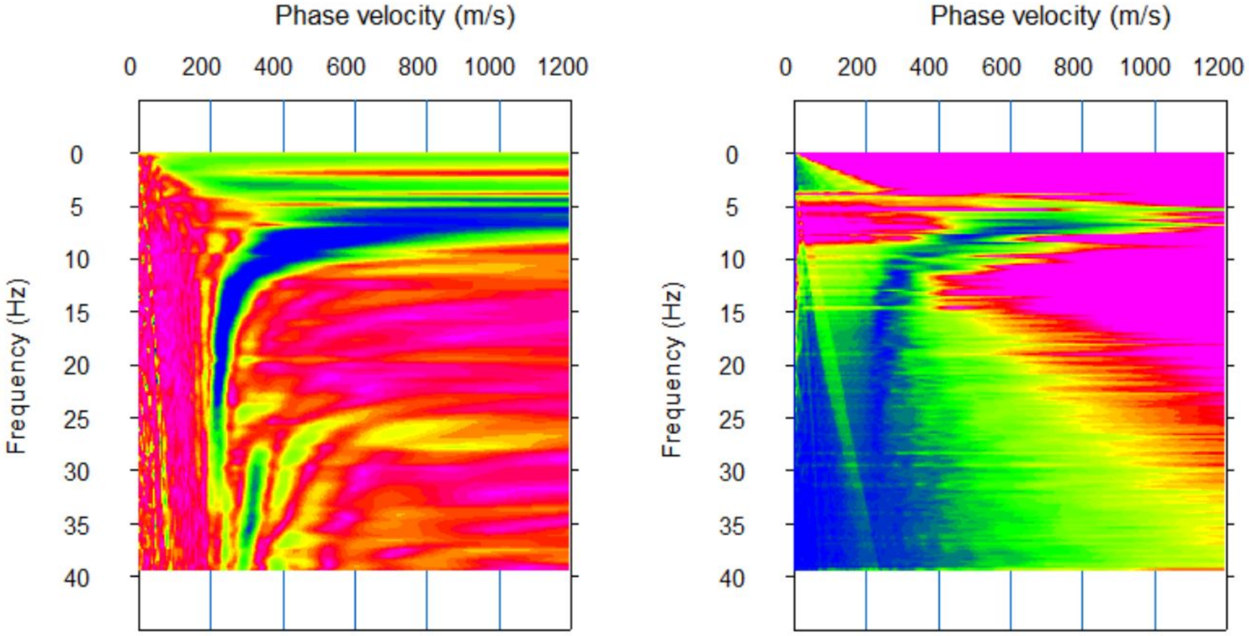


Figure 3 *Example shot record (top) and MASW dispersion images (bottom).*

The results of the MASW sounding are presented in Figure 4. The average shear wave velocity profile from the active shot records and passive data is plotted in the chart as a solid line. The dashed lines represent the upper and lower bound S-wave velocity profiles.

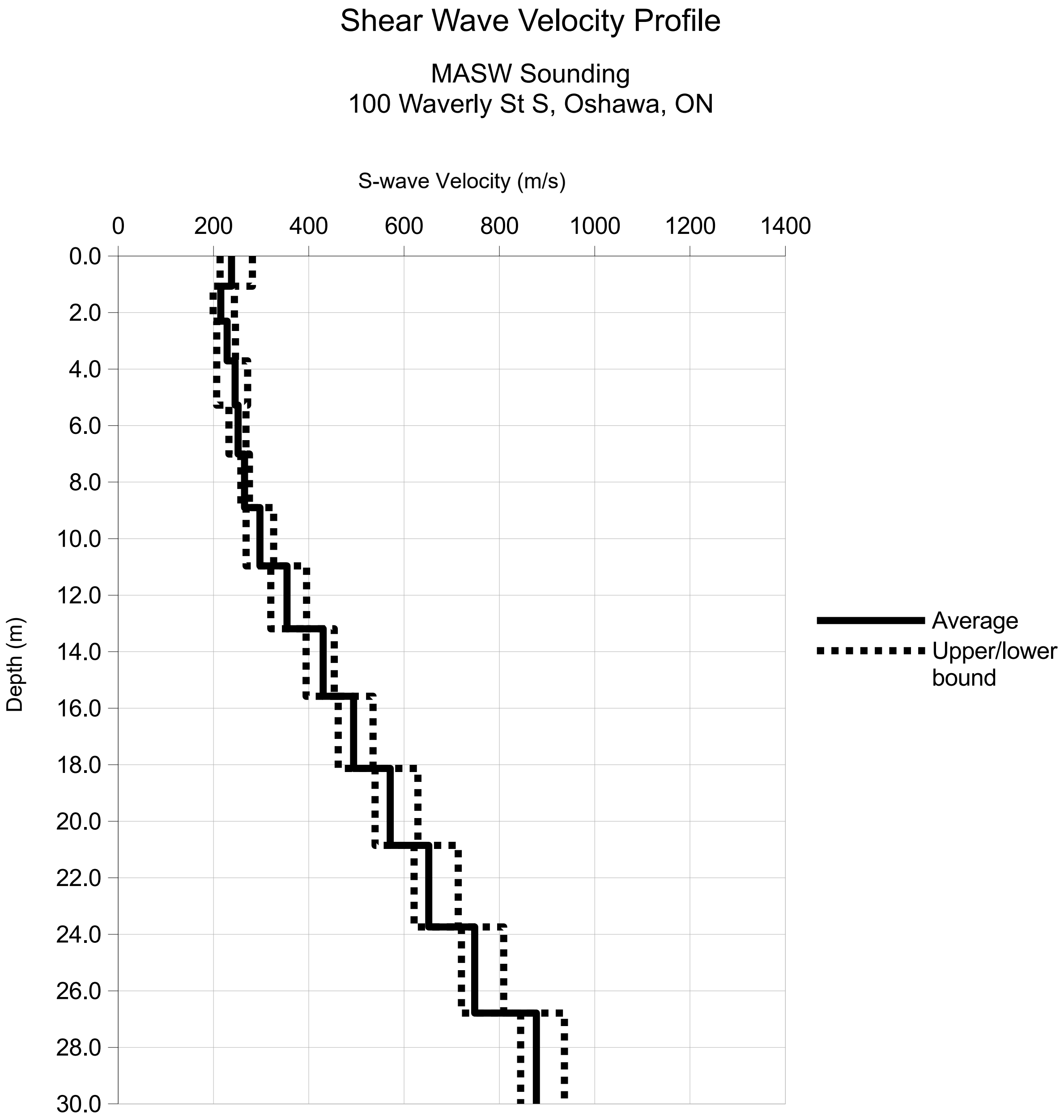


Figure 4 *Shear wave velocity profile from MASW sounding.*

The tabulated shear wave velocity model is presented in Table 1.

Table 1 *Shear wave velocities from MASW sounding.*

Depth Interval (m)		S-wave Velocity (m/s)
From	To	
0.0	1.1	238
1.1	2.3	215
2.3	3.7	229
3.7	5.3	245
5.3	7.0	252
7.0	8.9	265
8.9	11.0	298
11.0	13.2	354
13.2	15.6	430
15.6	18.1	494
18.1	20.9	571
20.9	23.7	652
23.7	26.8	748
26.8	30.0	878

The average shear wave velocity within the upper 30 meters (V_S30) is defined as the travel-time weighted average velocity from surface to a depth of 30 m and calculated using the following formula:

$$V_S30 = 30 / \Sigma (d/V_S),$$

where d is the thickness of any layer and V_S is the layer S-wave velocity. In other words, V_S30 is calculated as 30 m divided by the sum of the S-wave travel times for each layer within the topmost 30 m.

The calculated V_S30 values are presented in Table 2.

Table 2 *V_S30 values from MASW sounding.*

Depth Range (m)	Minimum V_S30 (m/s)	Average V_S30 (m/s)	Maximum V_S30 (m/s)	NBC 2020 Site Designation
0 to 30	356	**386**	421	**X_{386}**

The V_S30 values obtained from the MASW sounding varied from 356 m/s to 421 m/s with an average of 386 m/s.

Based on Sentence 4.1.8.4.(2b) of the National Building Code of Canada 2020 (NBC 2020), the **Site Designation** is **X_{386}**.

4 CLOSURE

Shear wave velocity testing involving the multi-channel analysis of surface waves (MASW) method was carried out for the proposed addition to Waverly Public School located at 100 Waverly Street South in Oshawa, Ontario.

The average shear wave velocity ($V_{S}30$) value calculated from in situ shear wave velocity measurements was **386 m/s**. Based on Sentence 4.1.8.4.(2b) of the National Building Code of Canada 2020 (NBC 2020), the applicable **Site Designation** is **X_{386}**.

We hope you find this report satisfactory. Should you have any questions or require additional information, please do not hesitate to contact the undersigned.

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