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September 16, 2026

Addendum No. 02

File Reference Number: RFP 2026 053

Title: Design-Build of New North Bay Shops Building

RE: Clarifications/Questions

QUESTIONS/CLARIFICATIONS:

Item 1: ONTC has identified that the Geotechnical Investigation Report referenced within the previously issued Technical Memo - Rock Probe Program dated April 22, 2026, was inadvertently omitted from the procurement documents. Upon review, ONTC determined that the referenced report, dated December 12, 2025, was a draft report. To ensure proponents are provided with the most current and complete geotechnical information available, ONTC is issuing the final Geotechnical Investigation Report dated May 21, 2026 in place of the draft December 12, 2025 report. The final Geotechnical Investigation Report is attached to this Addendum and shall be read in conjunction with the previously issued Rock Probe Technical Memo. Proponents shall consider both documents when preparing their Proposals.

This Addendum hereby forms part of the RFP.

Regards,

Nicole Laplante
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ONTC Office Building at 435 Whitson St.

North Bay, Ontario

Geotechnical Investigation

Ontario Northland

Final GI Report

May 21, 2026
02509305.001

ENGLOBE

Ontario Northland

435 Whitson St., North Bay, Ontario

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Revisions and publications log

REVISION No.	DATE	DESCRIPTION
0A	December 12, 2025	Draft Report Submitted for comment
1	May 21, 2026	Revised to Three-Storey, Final Issued

Distribution

1 PDF copy	Client

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Englobe Corp.'s subcontractors who have carried out on-site or laboratory work are duly assessed according to the purchase procedure of our quality system. For further information, please contact your project manager."

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APPENDICES

1 Introduction

As requested by the Ontario Northland (ONTC, Client), Englobe Corp. (Englobe) has carried out a geotechnical investigation to assess the soil conditions at the location of 435 Whitson St., North Bay, Ontario (see Key Plans, Drawings No. 1a and 1b, Appendix A). We have completed the field and laboratory testing programs and submit the results in this report along with our comments and recommendations.

The purpose of the investigation was to determine the subsurface soil and groundwater conditions at the site and, based on that information, prepare this engineering report with geotechnical recommendations pertaining to development of the site, including site grading, site servicing, potential foundations, and proposed building construction.

1.1 Site Conditions

The site is located at 435 Whitson St., North Bay, Ontario on the south side of the street and is set within a primarily industrial area of North Bay.

The site is currently developed with a two-storey structure formerly used for commercial purposes.

Underground utility service clearances were undertaken in advance of the investigation with natural gas noted on the north side of the property.

See Photo Essay, Appendix D.

2 Fieldwork

The geotechnical fieldwork for this geotechnical investigation was carried on November 4th, 2025. The fieldwork consisted of three (3) sampled boreholes (Borehole (BH) Nos. 1 to 3). The location of the boreholes are shown on the Borehole Location Plan, Drawing No. 2 in Appendix A.

The boreholes were advanced with a truck mounted drill rig operated by Limitless Drilling and equipped with continuous flight hollow stem augers and HQ size coring equipment. The field work was under the full-time direction of an experienced member of our engineering field staff who was responsible for logging individual borings, retrieving samples, field sample classification, plus overall field/drill supervision. At select boreholes, samples were obtained at frequent intervals of depth by using the Standard Penetration Test (SPT) method. The SPT method of sampling involves advancing a 50 mm outside diameter split spoon sampler with the force of a 63.5 kg hammer, freely dropping 760 mm, mounted in a trip (automatic) hammer. The number of blows per 300 mm penetration is recorded as the “N” value.

A supplemental rock probe program was undertaken on April 13, 2026 to provide general depths to bedrock.

All samples taken during this investigation were stored in labeled airtight containers for transport to our Englobe laboratory for visual examination and select laboratory testing. The routine laboratory testing consisted of natural moisture content determination and particle size analysis on select samples. Samples remaining following testing will be stored for a period of three months following the date of this report and then discarded unless otherwise instructed.

In order to comply with the intent of Ontario Water Resources Act Regulation 903 amended to O. Reg. 128/03, the boreholes were sealed with reverse augering techniques for the full depth and, where appropriate, the surface was sealed with a bentonite plug.

The borehole locations and elevations were established using a handheld GPS unit. These locations and elevations have not been confirmed by an Ontario Land Surveyor (OLS) and, as such, must be confirmed by an OLS prior to use in design.

All measurements in this report are in Metric units (unless otherwise noted).

3 Subsurface Conditions

Soil conditions are confirmed at the borehole locations only and may vary between borings. The boundaries between stratums indicated on the logs are inferred from non-continuous sampling, results of in-situ tests (i.e., SPT, etc.), observations during the drilling operations, and/or the response of the drilling equipment. These boundaries are approximations only and should not be regarded as exact planes of geological change as the actual transition may be gradual from one soil type to another. The description of compactness of the granular subsoils, in part, was based on the results of the SPT and/or the response of the drilling equipment. The consistency of very fine cohesive subsoils, if encountered, was based on in-situ vane tests. Refusal, if encountered, is defined as the point at which the augers and/or sampling equipment can no longer be practically advanced with the equipment used in this investigation. Defining the nature of auger refusal was outside the scope of work for this project.

3.1 Subsurface Summary Description

Detailed descriptions of the subsurface conditions revealed at the borehole locations are shown on the enclosed Record of Borehole Logs in Appendix B. The following is a brief description of the encountered subsurface conditions at this Site.

Table 3-1 Summary of Observed Stratigraphy at the Discrete Borehole Locations

Borehole ID	Approximate Depth (m)			
	Organic Sands	Sand	Gravel & Sand	Clay
BH No. 1	0.0 – 1.4	--	1.4 – 2.1	--
BH No. 2	--	0.0 – 0.3	--	--
BH No. 3		0.0 – 2.0 2.2 – 2.4	--	2.0 – 2.2

3.1.1 Organic Sands

A layer of mixed organics and sands, some gravel, trace silt was observed at BH No. 1. The thickness of the observed layer was 1.4 m, and the natural moisture content measured on samples obtained from this deposit ranged from 9 to 11%. Based on SPT ‘N’ values of 6 to 11 blows per 300 mm penetration; the compactness of this deposit was described as “loose to compact”. Gradation (sieve) analysis was carried out on two (2) samples obtained from this layer, the results of this test can be found in Table 3-2 below and are also summarized in Appendix C – Laboratory Test Results.

3.1.2 Gravelly Sand

A layer of gravelly sand, trace silt was observed underlying the organics and sands at BH No. 1. Occasional cobbles and/or boulders were observed within this deposit. The observed thickness of the gravelly sand layer was 0.7 m, and the natural moisture content measured on a sample obtained from this deposit was 13%. Based on an SPT ‘N’ value of 8 blows per 300 mm penetration; the compactness of this deposit was described as “loose”. Gradation (sieve) analysis was carried out on one (1) sample obtained from this layer, the results of these tests can be found in Table 3-2 below and are also summarized in Appendix C – Laboratory Test Results.

Auger refusal was encountered in this deposit in BH No. 1 at a depth of 2.1 m below grade.

3.1.3 Sand

At surface of BH Nos. 2 and 3 a layer of sand, trace to some gravel, trace to with silt was observed. The investigated thickness of the sand layer is between 0.2 and 2.0 m, and the natural moisture content measured on samples obtained from this deposit was between 7 to 61%. Based on SPT ‘N’ values of 4 to 7 blows per 300 mm penetration; the compactness of this deposit was described as “loose”. Gradation (sieve) analysis was carried out on two (2) samples within this, the results of these tests can be found in Table 3-2 below and are also summarized in Appendix C – Laboratory Test Results.

Auger refusal was encountered within this deposit in BH Nos. 2 and 3 at depths below grade of 0.3 and 2.4 m, respectively.

3.1.4 Clay

A thin layer of clay was observed sandwiched in the sand stratum at BH No. 3 at a depth of 2 m below grade. The estimated thickness of the clay layer was 0.2 m. No samples were obtained within this deposit.

3.1.5 Summary of Laboratory Test Results

The following section summarizes the laboratory data results obtained from relevant samples collected during the geotechnical investigation. Samples were obtained from the investigation at frequent intervals of depth by using the Standard Penetration Test (SPT) method.

The following laboratory tests were considered to determine relevant geotechnical information at select borehole locations:

- Natural Moisture Content

- Gradation (sieve and/or hydrometer)

Table 3-2, summarizes the gradation results (sieve) obtained from conducting laboratory testing on the following samples:

Table 3-2: Gradation Results

Borehole & Sample ID	Depth (m)	Material Type	Gradation			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH No. 1, SS 1	0.0 – 0.6	Organic Sands	18	78	4	
BH No. 1, SS 2	0.8 – 1.4	Organic Sand	2	93	5	
BH No. 1, SS 3	1.5 – 2.1	Sand & Gravel	43	50	7	
BH No. 3, SS 1	0.0 – 0.6	Sand	11	84	5	
BH No. 3, SS 3	1.5 – 2.1	Sand	14	60	26	

3.2 Groundwater Data

Groundwater levels and cave-in depths at the boreholes were measured, where possible, during the field investigation. It is noted that there may have been insufficient time for groundwater levels to stabilize while the boreholes were being excavated/open. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B) and the Table 3-3 below.

Table 3-3: Borehole Water Level Measurements

Boring ID	Ground Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)
BH No. 1	165.9	0.9	165.0
BH No. 2	166.1	Not Observed	--
BH No. 3	165.8	2.1	163.7

Groundwater levels will fluctuate seasonally and/or yearly. As such, the groundwater level must be established in advance of the construction operations (i.e. at time of tender or following award, prior to starting site work) such that adequate groundwater control plans can be developed

3.3 Supplemental Rock Probe Data

A rock probe program was undertaken on April 13, 2026 to provide general depths to bedrock, the details of which are included in Appendix F.

4 Discussion and Recommendations

The site is currently developed with a two-storey commercial style building. During recent renovations, exterior excavations revealed that the foundation walls appeared to be supported on a mud slab instead of a footing. Furthermore, an organic stratum was present below the footings which is likely a contributing source to the observed signs of movement in the building. Further to discussions on potential methods to correct the foundation conditions, such as underpinning, it was determined that the optimal solution was to demolish the existing building and build a new structure in the same location.

It is understood that the new building will be a slightly larger footprint but shifted some 2 m to the south, three storeys in height with a slab on grade with no below grade structures.

In general, the soil conditions at this site consists of some areas of surficial granular fill mixed with organics, underlain by loose sands. The groundwater is relatively shallow.

While shallow refusal was encountered at BH No. 2, it is reported that the Contractor undertaking the renovations had advanced test pits at each side of the building and these indicated that there is sufficient depth for shallow foundations. It is noted that shifting the building to the south, undulating bedrock may be encountered. A rock probe program identified variable depths to bedrock, see Appendix F.

The following discussions and recommendations are based on the factual data obtained from the investigation and are presented for guidance of the design professionals only. The comments pertain to a specific project and location. This report is provided on the basis of these terms of reference and on the assumption that the preliminary design features relevant to the geotechnical analyses will be in accordance with applicable codes, standards and guidelines of practice. If there are any changes to the site development features relevant to the interpretation made of the subsurface information with respect to the geotechnical analyses or other recommendations, then Englobe should be retained to review the implications of these changes with respect to the contents of this report.

Comments about construction are presented only to bring attention to aspects which might impact the design. Contractors bidding on or conducting work associated with this project should review the factual data presented in the preceding sections of the report, to assess their effect on proposed construction methods and scheduling.

4.1 Frost Protection

The estimated frost depth penetration for the area of the subject site, based on OPSD 3090.100 is:

- ± 1.9 m below exposed asphalt surfaces or for isolated, unheated foundations.
- ± 1.7 m for exterior footings in a heated structure below exposed surface (i.e., adjacent sidewalks, etc.);
- ±1.5 m for naturally insulated (i.e. snow cover) exterior footings for a heated structure.

The overburden soils at this site, within the anticipated depth of frost penetration, have a low to moderate to frost heaving.

At the locations where inferred shallow bedrock was found and the footings are founded directly on sound, unweathered bedrock, full depth frost protection is not required. This is provided that the geometry of the bedrock is such that groundwater flows away from the footings (i.e. groundwater will not pool adjacent to, or underneath the footings).

All exterior footings and isolated footings supported on soil and subject to frost penetration must have both permanent and temporary (during construction) frost protection to the depths noted above.

If a sufficient depth of earth cover cannot be provided for frost protection, equivalent synthetic insulation may be used in conjunction with available soil cover to provide frost protection. If synthetic insulation is used for frost protection, precautions must be taken to protect the insulation from accidental spillage of hydrocarbons, solvents, or other destructive products.

Foundations (including exterior slabs) can be founded at a higher elevation than required for frost protection provided they are supported on approved subgrades and insulated. The following general insulation design can be used. The following insulation design was based on the generalized design curves (Robinsky and Bespflug, 1973) for minimum insulation requirements for heated structures founded on sandy soil with a minimum of 300 mm of soil cover overtop the insulation.

For the heated foundation situation, synthetic insulation (i.e., Styrofoam SM, HI-40, HI-60, HI-100, depending on loading, or equivalent), minimum 50 mm thick, should be placed down the face of the foundation wall to the top of footing and then extend outwards horizontally beyond the outside foundation edge a minimum of 1.2 m.

Insulation at building corners should be doubled over 2 m from the corner. Beyond the foundation footprint, the horizontal insulation should be sloped downwards slightly (i.e., 2 to 3%) to promote drainage away from the structure. The insulation should be overlapped (or step jointed) and pegged or spot glued together. The insulation must be unbroken, and any damaged pieces must be replaced. The insulation should have a minimum of 300 mm of permanent soil cover. To reduce the risk of damage to the polystyrene insulation from an accidental hydrocarbon spill, it is recommended that the insulation be covered, where appropriate, with a layer of 6 mil polyethylene (i.e., maintenance areas, garage entrances, below parking lots, etc.). See Appendix E for Typical Sketches for frost protection.

Soils that are sensitive to frost heave may experience heave during the winter/spring months, only to settle back once thawed. As such, the founding subgrades for footings, slab on grade, services, etc. must be protected from frost penetration at all times during foundation excavation and construction operations. Should freezing temperatures occur during construction, the Contractor must undertake to prevent frost penetration into the natural soils (straw, insulated traps, portable heating etc.) until such a time that footings, slab on grade, services, etc. are adequately protected (soil cover, insulation, heat is supplied to the building, etc.).

Concrete cannot be placed against materials with sub-zero temperatures.

All granular backfill must be free of frost, ice, and snow, and at an appropriate moisture content and temperature to allow compaction. Once a lift of engineered fill is placed, compacted, and accepted, it is considered acceptable to backfill overtop of this lift if the lift is unfrozen or if there is minimal frost within the surface of the lift. If the surface of a granular fill lift is frozen, the Contractor shall, in conjunction with an Englobe representative, confirm depth of frost prior to backfilling. It is noted that frost penetration can be reduced through the use of insulated tarps, with or without heat source (depending upon ambient temperatures), and by ensuring backfilling operations are continuous.

In addition, active monitoring of the subgrade temperatures may be warranted depending upon the time of year that construction is undertaken. If winter construction is anticipated, a detailed winter construction plan shall be provided by the Contractor prior to the commencement of the project.

4.2 Foundation Recommendations

4.2.1 Shallow Foundations

4.2.1.1 General

Preliminary foundation design parameters are provided in this report for static, vertically, and concentrically loaded foundations in compression, unless otherwise specifically noted. Eccentric and other design parameters can be provided when more design details are available, if applicable and requested by the structural engineer. All foundation design recommendations presented in this report should be considered preliminary in nature for feasibility and volume considerations and subject to refinements and change during subsequent supplementary analysis during more detailed design stages of the project. In addition, all recommendations assume that an adequate level of construction monitoring during foundation excavation and installation will be provided. An adequate level of construction monitoring is considered to include:

- For shallow foundations, inspection of all excavation surfaces before engineered fill and foundations placement to ensure the suitability of the subgrade; and
- For earthworks, full-time monitoring and compaction testing for engineered fill below and above foundations.

It is recommended that the conventional shallow footings for the proposed structure be constructed of reinforced concrete and that the foundation walls be constructed of reinforced concrete dowelled into the footings, or masonry block, grouted and doweled into the footings.

Bearing areas will require careful preparation. Following excavation, all bearing surfaces should be cleaned of all organic, loose, disturbed, or sloughed material prior to forming footings and placing concrete. Bearing surfaces should be protected at all times from rain, freezing temperatures and the ingress of groundwater and/or surface water before, during, and after construction. Water control should be anticipated and planned for. All foundation excavations and bearing surfaces should be inspected by a qualified geotechnical engineer prior to forming footings to confirm the suitability of bearing surfaces and to confirm that the resistances provided in this report are consistent with what is observed during construction inspection.

Backfill above the footing area should consist of granular backfill such as OPSS.MUNI 1010 Granular B Type I compacted to at least 98% of the SPMDD.

4.2.2 Conventional Foundations Design - Soil Subgrade

Within the standard depth of conventional shallow reinforced concrete foundations, the subgrade below the organics will consist of loose sands. The natural subgrade is considered suitable, with some improvement, to support shallow foundations for a three storey lightly loaded (i.e. wood frame, etc.)

structures, however proper preparation of the subgrade will be critical to maintain the natural bearing capacity.

Noting the shallow groundwater table observed in the boreholes, dewatering in coarse-grained subgrades shall be planned for.

Excavations to the subgrade level shall be undertaken with equipment that produces a smooth subgrade surface, and does not result in deep shovel tooth gouges in the subgrade. Once prepared, the subgrade shall be inspected and approved by a qualified geotechnical consultant, and the subgrade proof-rolled to a consistent in-situ density. A pad of engineered fill shall be constructed below the footings, placed in the area of influence of the foundation.

It is recommended that footings bear on a granular pad of engineered fill, consisting of Granular A or Granular B Type II. The granular pad is to be a minimum thickness of 600 mm below the underside of the foundations and placed within the area of influence of the foundations. The imported Granular Type A or B Type II is to be compacted to 100% SPMDD.

The area of influence is described as a trapezoid below the foundation unit that extends to 300 mm to either side of the foundation and then down on 1H:1V slope to the natural subgrade.

As noted, shallow refusal was encountered at BH No. 2 on presumed bedrock. Bedrock is known to be present in the area at shallow depth, the surface of which can be very erratic and vary in elevation substantially in elevation over short horizontal distances. If bedrock is encountered, the Contractor shall ensure that sufficient bedrock is removed to allow for a minimum of 300 mm of engineered fill over the bedrock to act as a cushion. When the bedrock drops to lower elevation, the Contractor shall improve the subgrade as noted and increase the thickness of the engineered fill to the recommended 600 mm depth.

It is noted the control of the surface and groundwater will likely be required to maintain the available natural bearing and prevent subgrade disturbance during construction of the pad.

Shallow foundations can be designed using limit state static bearing pressures listed in Table 4-1 below. For these estimated bearing pressures to be realized, minimum soil cover is required above the footing as described previously. Adequate equivalent frost protection must be provided as per previously discussed, if footings are installed above the design condition frost depth. The geotechnical resistance of the proposed bearing areas can be estimated for the ultimate limit state (ULS) and serviceability limit state (SLS) for a maximum settlement of 25 mm and differential settlement of 19 mm. The geotechnical resistance at ULS was calculated by applying load resistance factor of 0.5 according to the 2023 Canadian Foundation Engineering Manual (5th Edition).

Table 4-1: Geotechnical Resistances and Reactions for Footings on Engineered Fill over Soil

Bearing Material	Depth of the footing, D (m)	Ultimate Bearing Capacity (kPa)	Factored Resistance at ULS (kPa)	Reaction at SLS (kPa)[1]
Minimum 600 mm of Engineered Fill over approved Natural Subgrade	0.5 to 1.0	350	175	100

Notes: (1) Subject to satisfactory site-specific settlement checks and necessary adjustments for each individual footing during detailed design stage

It should be noted that the footing dimensions have been assumed based on typical configuration for similar projects and information provided by the client. A minimum footing width of 600 mm is recommended for shallow foundation design and the above values assume a maximum footing width of 1.2 m.

Where unsuitable (e.g., peat/organic soil, existing fill and others) or unstable (e.g., disturbed during excavating, unexpected very loose or soft subgrade, etc.) soils are encountered during construction excavations, the foundation soils must be removed and replaced with compacted engineered fill to the foundation grade. The unsuitable material should be excavated, under the direction of a geotechnical engineer, to competent subgrade and then backfilled either with Granular 'A' material compacted to 100% standard Proctor maximum dry density (SPMDD) or with a lean concrete mix. The footprint of such removal of weak soils and replacement with engineered fill, should be considered on a case-by-case basis depending on depths required for removal so that bearing pressures can be distributed accordingly.

If soil conditions are encountered that vary significantly from those identified at the borehole locations, tapers between locations may also be required.

As previously stated, the engineered fill below footings should comprise Granular A or Granular B Type II material according to Ontario Provincial Standard Specifications (OPSS). The engineered fill should be placed and compacted in an unfrozen condition and the subgrade should be always protected from frost penetration. Engineered fill below footings shall be placed immediately upon excavation and following subgrade approval, using equipment compatible with lift thicknesses suitable based on-site conditions, and generally in level consistently placed lifts not exceeding 300 mm in thickness.

4.2.3 Shallow Foundations Supported on Bedrock

In consideration of the depth of potential frost penetration of up to 1.9 mbgs, it is likely that the inferred bedrock will be encountered at most, locations before the depth of frost penetration is achieved, noting again the erratic nature of bedrock surfaces in the area.

The bedrock at this site is favorable for support of shallow foundations for the proposed structure. However, considering the generally shallow refusal elevations identified on the site, bedrock removal may be required to allow for foundation construction at the desired design elevation. Further, as noted based on the bedrock cores, the upper portion of the bedrock may be weathered and fractured. Weathered and fractured bedrock shall be removed to sound bedrock surface.

Founding shallow footings on both a yielding (soil) and an unyielding (bedrock) subgrade is not recommended due to the potential for excessive differential movement between the two. Footings should all be founded on the same subgrade type. Given the presence of shallow bedrock and bedrock outcroppings on the site, some rock excavation under controlled conditions and possible bedrock grouting/sealing and/or lean concrete placement will be required to achieve the desired design elevation for the proposed concrete foundations. The design is also expected to include due considerations for the limited lateral resistance available at shallow depths for the proposed foundation and the use of properly designed rock anchors (if needed).

Organics, fill, and other deleterious materials including fractured/weathered bedrock must be removed from the area of influence of the foundations down to sound bedrock. All organic and fill material is required to be stripped from the building footprint prior to the start of the foundation construction.

Footings can be founded directly on the sound bedrock surface. No footing is permitted to bear directly on the highly fractured upper bedrock. This will require that all loose materials or weathered/fractured bedrock be removed from below the footing. Concrete (minimum 20 MPa) may be used to achieve a level surface for the proposed foundation. Bedrock subgrade preparation requires the following procedures under the direction of the Geotechnical Engineer:

- Sub-excavate any zones of weak rock.
- Clear all bearing surfaces of soil and loose rock as identified by excavator equipment.

- Assess any open or soil-filled joints. Unless advised otherwise by the Geotechnical Engineer, treat by cleaning cracks and joints to a minimum depth equal to 3 times the width and filling with cement mortar or slush grout, dependent on crack width.
- Apply concrete to level out rock surface sufficiently for uniform foundation support.

As noted, for footings supported directly on sound bedrock, the full depth of soil cover required for frost protection is not necessary. If any bedrock surface supporting foundations slopes greater than 10° off the horizontal, dowels may be required to resist sliding and a qualified geotechnical engineer must review the conditions in the field. Alternatively, the contractor could level the footing bearing surface with a hydraulic hammer (hoe-ram), pouring concrete or other equivalent means. The contractor must be prepared to adjust his operations to address the anticipated variable conditions. The following provides the factored geotechnical resistances for footings established directly on sound bedrock:

Table 4-2: Geotechnical Resistances and Reactions for Footings on Bedrock

Depth to Inferred Sound Bedrock (m)[1]	Resistance at Factored ULS (kPa)	Reaction at Factored SLS (kPa)
1.0 – 1.5	5,000	N/A [(1)]

Note:
(1) Assumes upper 400 mm of bedrock on average to be weathered/fractured based on local data.
(2) Since bedrock is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply.

Rock Anchors

The resistance to the lateral and overturning forces can be from the backfill material above the footing and the bearing of the rock below the footing. In consideration of the anticipated shallow bedrock at this site, if there is insufficient earth cover to provide lateral/uplift resistance, rock anchors can be used to provide the required lateral and uplift resistance.

Recommendations for Anchors:

i. Anchor Type should be non-shrink grout anchors.

ii. Assuming non-weathered rock, moderately fractured, an allowable working bond stress at the rock-grout interface should not exceed 1,000 kPa.

iii. The upper 500 mm of bedrock from the bearing surface should be ignored in determining the anchor bond capacity.

iv. A maximum apex angle of 60 degree has been assigned for determining the cone of rock mobilized by the anchor.

v. Based on local site data, the upper bedrock is highly fractured and of poor to excellent quality, but shall be assumed to be of good quality at this Site. Performance testing of anchors on test anchors to verify the design capacities of the materials used before the actual anchors for the foundations are installed are an essential requirement. Anchors installed for the new building should be tested to include a selection of performance and proof testing in general accordance with ASTM D4435 – Rock Bolt anchor pull test, and guidelines set forth in the Post Tensioning Institute documents to ensure the anchors have meet the project requirements.

vi. Anchor design should also take into consideration the loading direction where loads may not be normal to the rock surface or parallel to the anchor alignment.

Mechanical rock anchors are generally not recommended under these conditions (highly fractured bedrock) unless installed to sufficient depth with representative proof testing. The grouted anchors noted above have the advantage of locally filling fractures within the anchor column with grout, thereby solidifying localized areas of the bedrock. However, as mechanical anchors are proprietary system, Suppliers may suggest mechanical anchors. In this case, the Supplier should be provided with the factual information so that an appropriate anchor system can be designed, following which, a sufficient number of the anchors must be proof tested to confirm their design.

4.2.4 Typical Soil Parameters

The geotechnical soil design parameters are summarized in Table 4-3. Typical soil parameters are provided for specified types of manufactured fills per OPSS. In addition, based on the results of in-situ and laboratory tests conducted to date and presented in Appendix C, parameters are suggested for use in design for the soil types encountered at the boreholes.

Table 4-3: Suggested Soil Parameters for Geotechnical Design Analyses

Soil Type	Unit Weight(kN/m^3)[1,2]	Angle of Internal Friction, ϕ (Degree)[1,2]	Interface Friction Angle,δ (Degree)[1,3]
Engineered Fill (Granular A)	22	34	22
Engineered Fill (Granular B Type I)	19	31	17
Engineered Fill (Granular B Type II)	21	35	21
Existing Sand	20 to 21 (20)	28 to 33 (30)	16

Note:

(1) Recommended parameters have been estimated based on visual observation of the soil conditions, results of measured field testing, laboratory test results, correlation with published information (Terzaghi, Peck, and Mesri, Third Edition; Kenney, 1959; Ohsaki et al. 1959; CFEM, 5th Edition) and our previous experience with similar materials.

(2) Design values are in brackets

(3) Interface between soil and concrete, CFEM, 5th Edition, Table 20.4

4.2.5 Lateral Earth Pressures

Temporary bracing and shoring may be designed using the typical soil coefficients and parameters given in Table 4-4, however the designer/contractor should verify the appropriate soil parameters for the designs of a specific bracing and shoring system. The design should incorporate the effects of hydrostatic pressure, traffic surcharge and retained sloping earth conditions in the bracing design. The following parameters may be used for design. The parameters are based on general representative values for the various soil types, obtained through laboratory testing and tactile analysis and are presented in the table below.

Table 4-4: Lateral Pressure Coefficients

Parameter	Granular A	Granular B Type I	Granular B Type II	Existing Sand
Angle of Internal Friction	34°	31°	35°	30
Coefficient of Active Earth Pressure (Ka)	0.28	0.32	0.27	0.33
Coefficient of Passive Earth Pressure (Kp)	3.54	3.12	3.69	3.00
Coefficient of Earth Pressure at Rest (Ko)	0.44	0.48	0.43	0.50

4.2.6 Shallow Foundation Sliding Resistance

The sliding resistance can be calculated using the following formula:

$F_r = \Sigma \mathrm{W} \tan \delta$

Where,

F_r = base resistance to sliding (ultimate).

δ = Interface friction angle (use Table 4 2)

ΣW = Total weight of the of vertical forces acting on footing.

A resistance factor of 0.8 should be applied to the ultimate sliding resistance in accordance with Canadian Foundation Engineering Manual (5th Edition).

4.3 Slab-on-Grade

Slab on grade construction can be used at this site. All deleterious materials (i.e., fill, organics, etc.) should be removed from below the slab on grade. The slab on grade shall be built on new granular engineered fill.

The Contractor should be prepared to locally excavate deeper to remove unacceptable areas that may become apparent during construction operations. A minimum of 300 mm of engineered fill shall be placed below the slab on grade however, as noted, the organics must be removed. The subgrade must be approved by a qualified geotechnical consultant.

The approved subgrade can then be brought up to the underside of the vapour retarder with engineered fill consisting of an imported well-graded coarse grained granular material meeting OPSS.MUNI 1010 for Granular B Type III or Granular A, compacted to a minimum 98% Standard Proctor Dry Density (SPDD) or better. A well-graded coarse-grained soil is described as having no excess particles in any size range with no intermediate sizes lacking (i.e., smooth, concave distribution curve). Generally, the distribution curve for this backfill should fall in the middle of the specification and be limited to maximum sized particles of 75 mm or less (to prevent damage when backfilling against underground structures) and should contain at a minimum 2% fines to facilitate compaction efforts. The use of a well-graded material will facilitate compaction operations.

The use of a vapour retarder will be dependent upon the floor coverings to be used and the floor covering manufacturers' recommendations should be followed. For preliminary design, a manufactured vapour retarder system (i.e., min 10 mil polyethylene, etc.) over top of the engineered fill may be used. The vapour retarder manufacturer's specifications must be adhered to (minimum overlaps, taping/sealing at openings, sealed around utility and foundation or column perforations, etc.) and the integrity of the system must be maintained (i.e., no holes, tears, or other perforations). The concrete supplier and finisher should undertake to use a mix and placement methodology that will minimize the potential for slab curling.

To achieve a reasonable level of performance from a grade-supported floor slab, it is essential to have a relatively uniform subgrade. Cracking, differential movements, and poor performance of floor slabs may be related to variations in the subgrade support. Uniformity in material, moisture content and density would be required. This level of uniformity would require the same type of material throughout the entire subgrade, placed at a similar moisture content and density.

The slab should be structurally independent from walls and columns which are supported on foundations. This is to reduce any structural distress that may occur as a result of differential soil movement. If it is intended to place any internal non-load bearing partitions directly on the slab-on-grade, such walls should also be structurally independent from other elements of the building founded on the conventional foundation system so that some relative vertical movement of the walls can occur freely.

The subgrade beneath slab-on-grade shall be protected at all times from rain, snow, freezing temperatures, excessive drying and the ingress of water. This applies during and after the construction period.

Some relative movement between floor slab-on-grade and adjacent walls or foundation and differential movement within the slab should be anticipated. Generally, if the recommendations outlined in this report are followed, these movements are estimated to be less than 10 mm.

4.4 Drainage

It is understood that there will be no below grade structures and, as such, full perimeter foundation drains and underslab drainage should not be necessary provided the underside of slab on grade is a minimum of 150 mm above exterior grade.

The exterior foundation backfill should extend a minimum lateral distance of 600 mm out from the foundation wall and should consist of free-draining granular material, such as a Granular B Type I (OPSS.MUNI 1010) or suitable alternative drainage cellular media.

4.5 Earthquake Parameters

The 2024 Ontario Building Code (2024 OBC) stipulates the methodology for earthquake design analysis, as set out in Subsection 4.1.8.7. The determination of the type of analysis is predicated on the importance of the structure, the spectral response acceleration and the site classification. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4B of the 2024 OBC. The classification is based on the determination of the average shear wave velocity in the top 30 metres of the site stratigraphy, where shear wave velocity measurements have been taken or alternatively estimated based on rational analysis of undrained shear strength or penetration resistance.

At this site, it is known the upper soil stratigraphy consists of loose sands underlain by bedrock.

Considering the geotechnical values, and based on 2024 OBC, Table 4.1.8.4B, Site Classification for Seismic Site Response, the subject site would have Site Class C.

Consideration may be given to conducting a site-specific Multichannel Analysis of Surface Waves (MASW) at this site to determine the average shear wave velocity in the top 30 metres of the site stratigraphy. An improved seismic site designation (Site Class A or B) may be possible.

For seismic design purposes, Site Classification for Seismic Site Response, the subject site would have Site Class C (Site Designation X_c), for all footings founded on native soil. Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV) values were estimated using web-base software tool (https://seismescanada.rncan.gc.ca/hazard-alea/interpolat/index-en.php) from Natural Resources Canada. The output from the software for the site classifications is provided in Appendix E. The calculations are based on the approximate location of the Site (Latitude 46.301° and Longitude -79.453°).

The design peak ground acceleration (PGA) and Peak Ground Velocity (PGV), for foundation placed on engineered fill pad over undisturbed native soils (mainly sands), were calculated as 0.198 g and 0.160 m/s, respectively, with a 2% probability of exceedance in 50 years based on the interpolation of the 2020 National Building Code Canada (2020 NBCC) Seismic Hazard calculation.

4.5.1 MASW

A MASW was carried out on the same site within 100 m of the subject site, and this information can be used for design. The MASW study is included in Appendix E.

4.6 Excavation and Dewatering

Based on the Occupational Health and Safety Act Regulations for Construction Projects (O.Reg. 213/91, s. 226(4)), the existing fills and native soils are classified as follows:

- Native sands – Type 3

All excavations greater than 1.2 m in depth must be sloped or shored in accordance with the Occupational Health and Safety Act Regulations for Construction Projects. Short-term (i.e., day) open excavations will be stable above the groundwater table at a temporary angle of 1H:1V, however excavations established at this slope must not be left unattended at any time. Below the prevailing groundwater table, the slopes of open excavations will have to be flattened to 3H:1V or possibly shallower depending upon the method of dewatering employed or possibly sheeted.

When approaching the founding soil subgrade surface, the excavating Contractor should use equipment that will not leave deep gouges in the bearing surface. If there are tooth gouges in the subgrade, these are indicative of disturbance and can collect water, further affecting the subgrade.

Excavations must be maintained in a dewatered condition during excavation and foundation construction, and every reasonable effort must be made to prevent disturbing (piping/boiling) at the founding subgrade. Groundwater control, in accordance with OPSS.MUNI 517 and 518, will be required to maintain a stable subgrade during excavation and backfilling operations. The Contractor must undertake to establish the groundwater level in advance of the construction operations such that adequate groundwater control plans can be developed.

Groundwater was observed as shallow as 0.9 m below grade within the depth of the boreholes at the time of the investigation. There likely was insufficient time for the groundwater to stabilize within the boreholes prior to backfilling.

Depending upon the final depth of foundation elements, typical localized dewatering during construction, such as installation of a sufficient number of filtered sumps and pumping from these sump holes established below the base of the excavation may be required to maintain the excavation in a dewatered condition during subgrade preparation. The effectiveness of this method of groundwater control would be limited to conditions where excavations of less than 0.5 m below the prevailing groundwater table are anticipated, and the soil is such that the groundwater can be drawn down a minimum of 0.5 m below the working surface. If the excavation must penetrate to a greater depth below the prevailing groundwater table, a more effective groundwater control method such as a vacuum well point system with or without a sheet pile cut-off wall, should be considered by the contractor to maintain a stable excavation base. The Contractor's dewatering method must be designed to prevent piping.

Ultimately, the method of dewatering will be the choice of the Contractor. The importance and benefits of maintaining a dry stable subgrade during excavation and foundation construction cannot be stressed enough. Failure by the contractor to adequately control the groundwater, and/or rainwater, surficial runoff, etc., can result in disturbance to the founding subgrades, which can result in having to carry out corrective measures (i.e., additional excavation, time delays, etc.) to improve the subgrade. Corrective measures required to improve subgrades where groundwater is not adequately controlled will be at the Contractors cost. As part of the Contractors proposed methodology of construction, the Contractor should be requested to submit a dewatering plan prior to commencement of the project that details how they will control groundwater. The plan should include all aspects from methodology (e.g., sump holes and pumps, drainage ditches, vacuum well points), to construction of system (sump hole details, placement, etc.), to operation of system, etc.

The EPA requires a person who is engaging in the prescribed water taking activities set out in O. Reg. 63/16, that meet the criteria set out in that regulation, to register those activities in the Environmental Activity and Sector Registry (EASR) and possibly obtain a Permit to Take Water (PTTW). As of July 1, 2025, proponents may self-register dewatering activities under EASR regardless of the volume of water taken.

The soils within the anticipated depths of excavation, mainly sands, were not found to meet any OPSS.MUNI 1010 specification and can therefore only be used in areas of landscaping or elsewhere where movement of the ground surface is not of concern.

Any soil to be removed from the Site may be considered excess soil and is subject to O. Reg. 406/19: On-Site and Excess Soil Management.

Any granular material to be used as engineered fill on this site must be tested and approved by this office prior to delivery to the site. It should be noted that engineered fill(s) should be placed in lifts of thickness less than the effective compaction depth of the equipment used to carry out the compaction operations (i.e., if using a heavy diesel Wacker, lifts should be a maximum of 300 mm thick, etc.).

4.6.1 Excavations in Bedrock

It is noted that shallow auger refusal was encountered at the borehole locations. When encountered, the underlying natural bedrock surfaces in this area can be very erratic in nature, varying substantially in elevation over short horizontal distances. Surficial bedrock is visible throughout the site in various areas. As such, it is possible that bedrock removal may be required to allow construction.

Bedrock excavation (e.g., blasting) is likely to be required at this site and reference shall be made to OPSS.MUNI 120. A blast design is required to be provided by the blasting contractor before blasting operations are carried out. A pre-blast survey (OPSS.MUNI 120.07.03) shall be prepared for all buildings, utilities, structures, water wells, and facilities likely to be affected by the blast and those within 150 m of the location where explosives are to be used. Blast monitoring is to be undertaken during each blast, by an independent blast monitoring consultant retained on behalf of the owner. During each blast, ground vibration PPV and the peak sound pressure level shall be monitored 100 m from the blast or at the closest portion of any residence, utility, structure, or facility within this radius (OPSS.MUNI 120.07.05). The ground vibration PPV must be kept below 15 mm/s. Results are to be provided to the client's representative daily.

Other conventional methods such as drilling and hoe ramming techniques, the use of expanding chemical compounds, or mechanical rock splitters can also be used for the purpose of fragmenting rock. After loosening of bedrock, the bedrock can be removed by using excavators, backhoes, or other suitable mechanical excavation equipment. Suitable excavation equipment shall be determined by a qualified contractor.

Temporary excavation in bedrock may be carried out at near vertical slopes, provided the trench sides are cleared of loose rock prior to workers entering the trench. If the bedrock is overly fractured such that the loose rock cannot be entirely removed, a temporary rock catchment system such as a wire mesh system should be used. The catchment system should be designed to contain and/or prevent loose rock particles from falling on workers within the excavation.

Excavation work shall comply with the Occupational Health and Safety Act regulation – O.Reg. 213. Groundwater control is required for excavation work.

4.7 Monitoring During Construction

All foundation design recommendations presented in this report assume that an adequate level of construction monitoring by qualified geotechnical personnel during construction will be provided. An adequate level of construction monitoring is considered to be:

a) For shallow foundations: design review and full-time monitoring during construction; and

b) For earthworks: full-time quality control and compaction testing.

An important purpose of providing an adequate level of monitoring is to check that recommendations, based on data obtained at discrete borehole locations, are relevant to other areas of the site.

To provide an adequate level of construction monitoring, qualified geotechnical personnel should manage and supervise the following tasks during construction:

Shallow and Deep foundations:

- Confirm that materials and methods meet specifications.
- Inspect foundation subgrades.
- Inspect excavation.
- Review shallow foundation installation/testing methods.
- Review compaction testing records.
- Provide review comments, including any discrepancies found with respect to specifications as well as this report, and the need for any modifications to the design or methods.

Earthworks:

- Confirm that materials and methods meet specifications.
- Inspect subgrade prior to any fill placement.
- Quality control of granular, select fill material, and hot mix asphalt.
- Review compaction testing records.

5

5 Limitations

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations. If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that Englobe will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should Englobe not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

The professional services provided for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise stated specifically in the report. The recommendations and opinions given in this report are based on our professional judgment and are for the guidance of the Client or its Agent in the design of the specific project. No other warranties or guarantees, expressed or implied, are made.

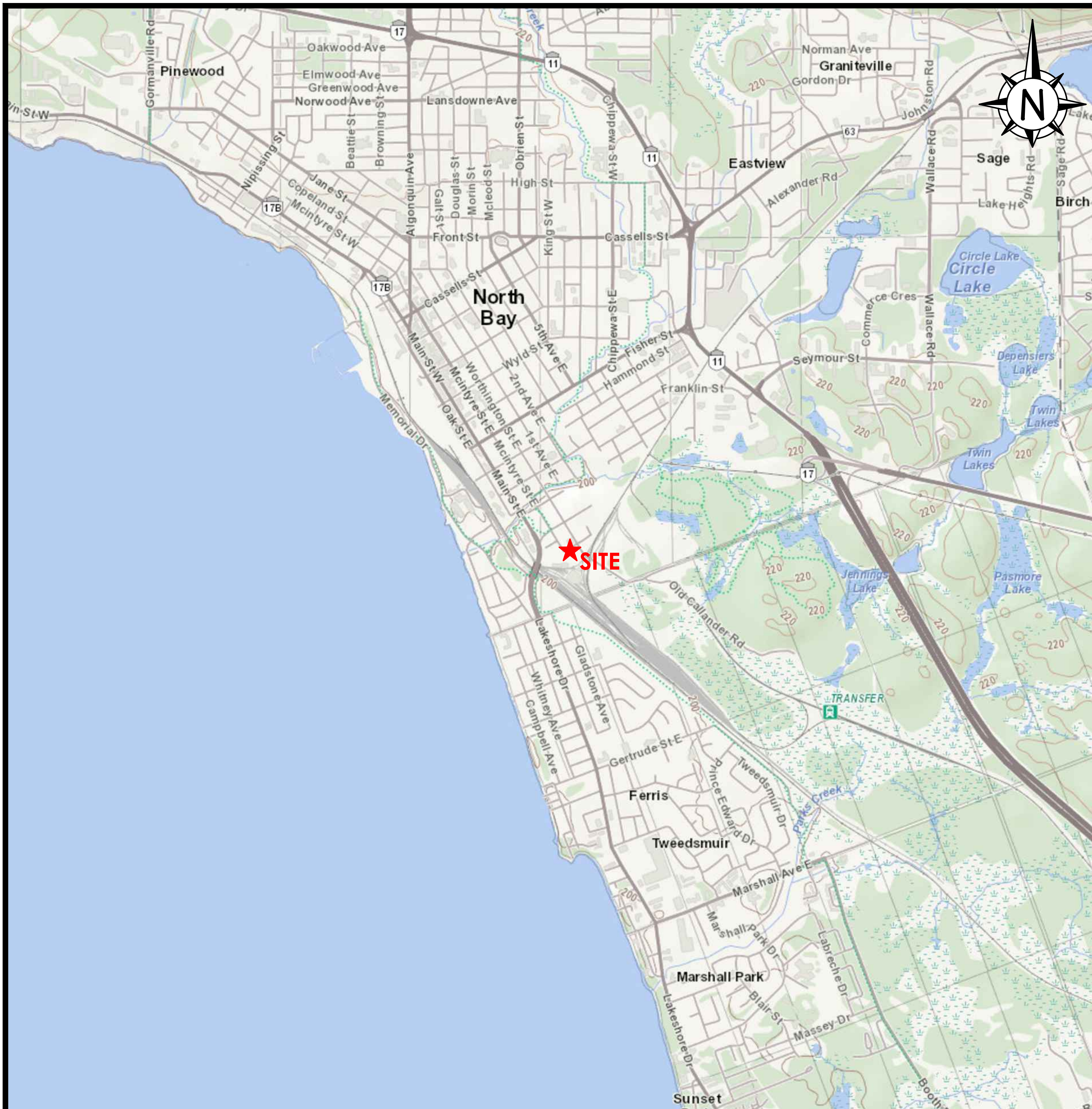
The comments in this report are intended solely for the guidance of the design team and address the geotechnical conditions only. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Inclusion of the factual information (Sections 1 to 3 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 4 of this report is intended solely for the use of the client and the design team. If this section is provided to the Contractor, it is solely to provide an understanding of the geotechnical aspects of the site, and alternatives presented are not to be considered potential substitutes of the final design. If there is a discrepancy between this report and the tender documents and/or construction drawings, the latter shall govern and the discrepancy must be immediately brought to the attention of the design team.

Appendix A
Drawings

Drawing No. 1a and 1b	**Key Plans**
Drawing No. 2	**Borehole Location Plan**

ENGLOBE



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No.	Version	Date	By	Verif	Appr.
00		2025/11/07	DM	JRB	JRB

ONTC

Geotechnical Investigation
435 Whitson St.
North Bay, ON

Key Plan (Macro)

ENGLOBE

2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical		Prepare by: DM	Verify by: JRB
Scale: Not To Scale		Draw by: DM	Approval by: JRB
Date: 2025/11/07		Drawing no: 1a	
Page setup: Macro	Paper size: 8.50 X 11.00 in.	Register no.: ----------	

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
JRB	02509305	001	GE	-	--	--	----	00

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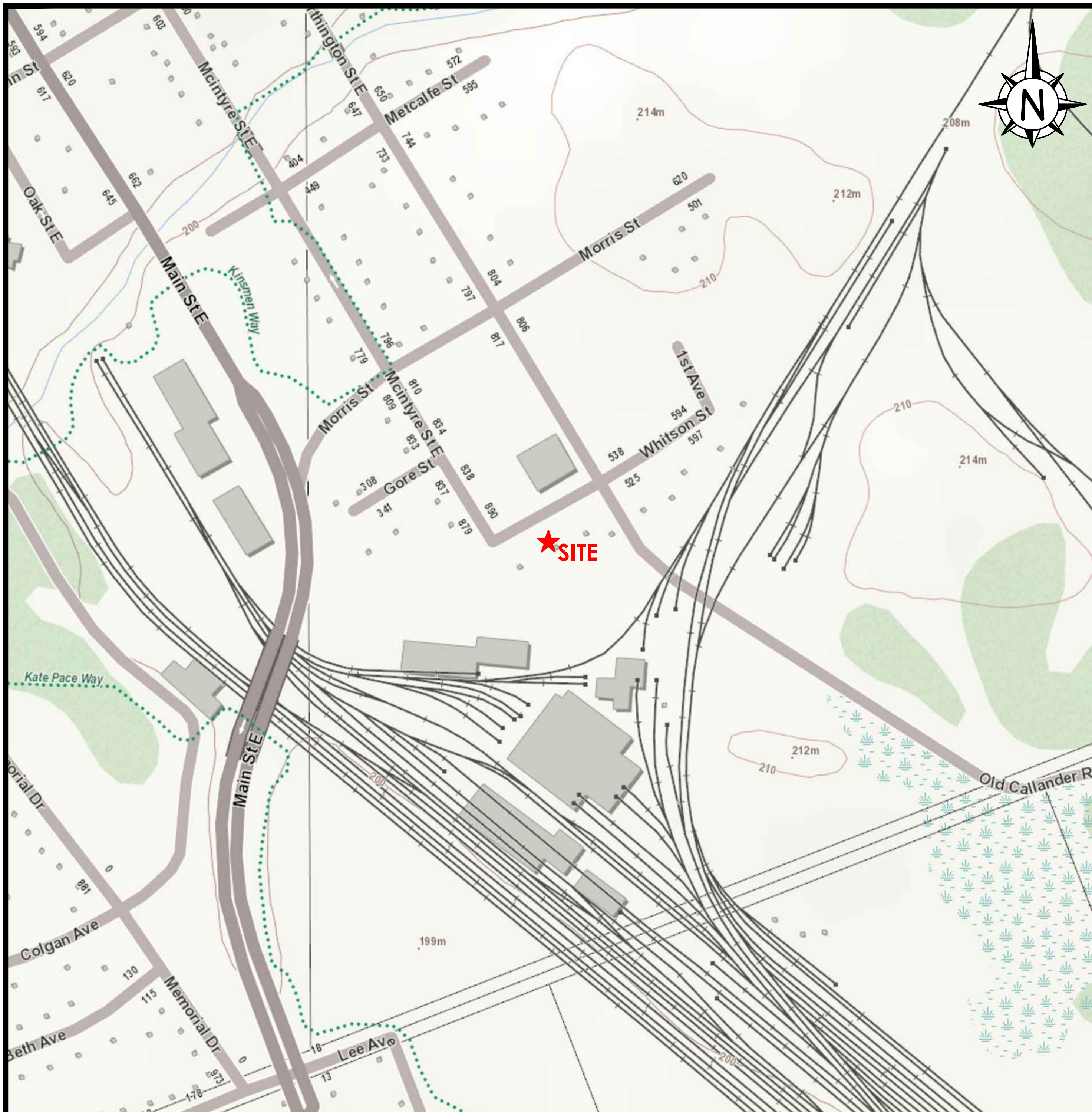

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No.	Version	Date	By	Verif	Appr.
00		2025/11/07	DM	JRB	JRB

ONTC

Geotechnical Investigation
435 Whitson St.
North Bay, ON

Key Plan (Micro)

ENGLOBE

2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical		Prepare by: DM	Verify by: JRB
Scale: Not To Scale		Draw by: DM	Approval by: JRB
Date: 2025/11/07		Drawing no: 1b	
Page setup: Micro	Paper size: 8.50 X 11.00 in.	Register no.: ----------	

Man.	Project	Otp	Project	Phase	Electronic ref.			Rev.
JRB	02509305	001	GE	-	--	--	----	00

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No.	Version	Date	By	Verif	Appr.
00		2025/11/07	DM	JRB	JRB

ONTC

Geotechnical Investigation
435 Whitson St.
North Bay, ON

Borehole Location Plan

ENGLOBE
2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical	Prepare by: DM	Verify by: JRB	
Scale: Not To Scale	Draw by: DM	Approval by: JRB	
Date: 2025/11/07	Drawing no: 2		
Page setup: BH Plan / Paper size: 8.50 X 11.00 in.	Register no.: ----------		

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
JRB	02509305	001	GE	-	--	--	----	00

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Appendix B
Borehole Logs

Enclosure No. 1	**List of Abbreviations and Symbols**
Enclosure No. 2	**Record of Borehole Sheets**

ENGLOBE

LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as ——•——•——•——•

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	*Compactness Condition*
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

3. SOIL DESCRIPTION (Cont'd)

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	*Consistency*
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

c) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

+ 3.2 - Field Vane test in borehole. The number denotes the sensitivity to remoulding.

D - Laboratory Vane Test

¨ - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

5. LABORATORY TESTS

P	Standard Proctor Test
A	Atterberg Limit Test
GS	Grain Size Analysis
H	Hydrometer Analysis
C	Consolidation

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as ‘fill’. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.

2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.

3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled ‘possibly’ or ‘probable’ etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients’ request, through a series of closely positioned unsampled auger probes combined with core drilling.

4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

ENCLOSURE NO.:2 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 1

ENGLOBE

REFERENCE 02509305.001 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg No. 2 ORIGINATED BY KG

PROJECT 435 Whitson St., North Bay BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY DM

CLIENT ONTC DATE (Started) 4 November 2025 DATE (Completed) 4 November 2025 TIME (Completed) CHECKED BY JRB

ELEV / DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
165.9	Ground Surface										
0.0	ORGANIC SANDS - some gravel, trace silt brown (loose)		1	SS	6				○		18 78 (4)
	trace gravel, trace silt (compact)		2	SS	11	∇	165		○		2 93 (5)
164.5 1.4	SAND & GRAVEL - trace silt occasional cobble/boulder encountered very wet (loose)		3	SS	8		164		○		43 50 (7)
163.8 2.1	Auger Refusal End of Borehole										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 4/11/25 1:30:00 PM	0.9	1.2
2)	-	-
3)	-	-

MEL-GEO 02509305.001 - BOREHOLE LOGS, 435 WHITSON.GPJ MEL-GEO.GDT 3/12/25

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

ENCLOSURE NO.:3 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 2

ENGLOBE

REFERENCE 02509305.001	DATUM TBM	LOCATION See Borehole Location Plan; Appendix A, Dwg No. 2	ORIGINATED BY KG
PROJECT 435 Whitson St., North Bay		BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers	COMPILED BY DM
CLIENT ONTC		DATE (Started) 4 November 2025 / DATE (Completed) 4 November 2025 / TIME (Completed)	CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
166.1	Ground Surface										
0.0	150 mm organic soil										
	SAND - trace gravel, trace silt		1	SS	52/150 mm		166				
165.8											
0.3	Spoon Refusal End of Borehole										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.001 - BOREHOLE LOGS, 435 WHITSON.GPJ MEL-GEO.GDT 3/12/25

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

ENCLOSURE NO.:4 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 3

ENGLOBE

REFERENCE 02509305.001 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg No. 2 ORIGINATED BY KG

PROJECT 435 Whitson St., North Bay BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY DM

CLIENT ONTC DATE (Started) 4 November 2025 DATE (Completed) 4 November 2025 TIME (Completed) CHECKED BY JRB

ELEV DEPTH (m)	DESCRIPTION (see Enclosure No. 1)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
165.8	Ground Surface						
0.0	50 mm asphalt SAND - medium to fine, some gravel, trace silt brown (loose)	1	AS			o	11 84 (5)
	some to trace gravel, trace silt brown to grey	2a	SS	7		o	
		2b				o	
	trace gravel, with silt	3	SS	4		o	14 60 (26)
163.8 2.0	CLAY - trace silt grey				∇		
163.6 2.2	SAND - some gravel, with silt very wet	4	SS	>50/0 mm			
163.4 2.4	Auger Refusal End of Borehole						

Elevation scale: 165, 164

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

	Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	4/11/25 3:00:00 PM	2.1	2.2
2)		-	-
3)		-	-

MEL-GEO 02509305.001 - BOREHOLE LOGS, 435 WHITSON.GPJ MEL-GEO.GDT 3/12/25

Englobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@englobecorp.com

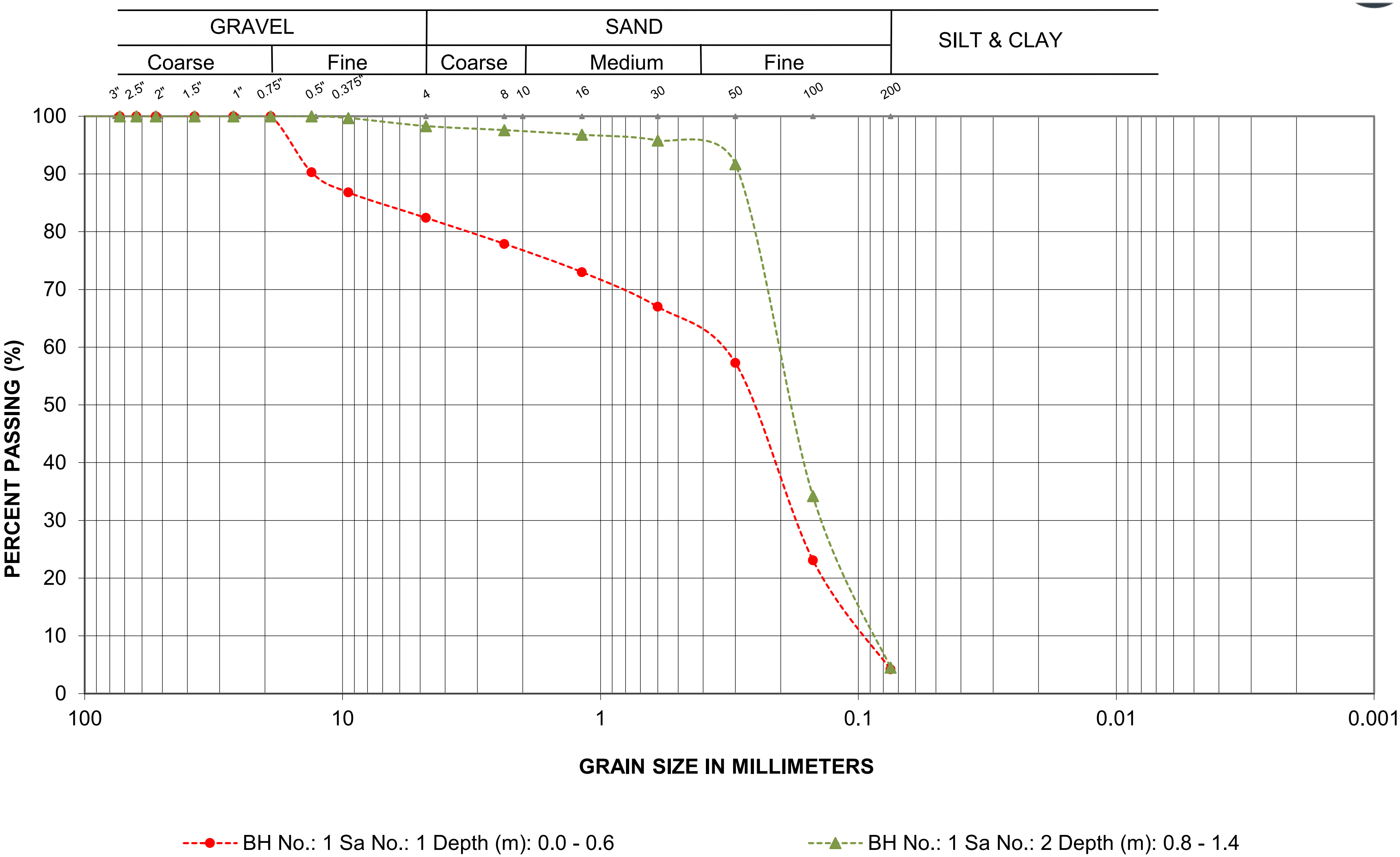
Appendix C
Laboratory Test Results

Lab Data

ENGLOBE

Reference No.: 02509305.001
Date: November, 2025

GRAIN SIZE ANALYSIS



ORGANIC SANDS

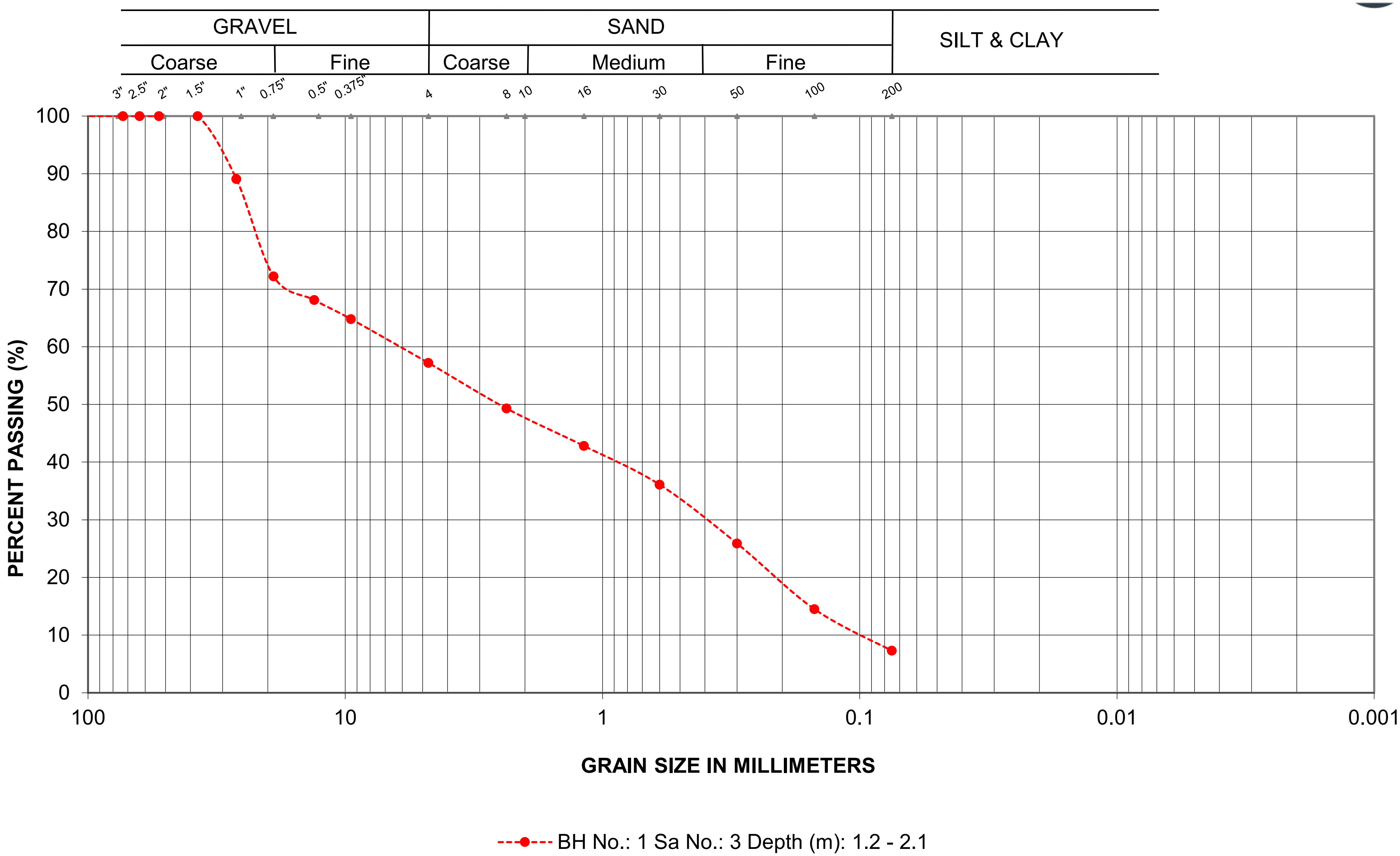
PROJECT: 435 Whitson.
LOCATION: North Bay, ON

Englobe Corp.

FIGURE L-1

Reference No.: 02509305.001
Date: November, 2025

GRAIN SIZE ANALYSIS



GRAVEL SAND

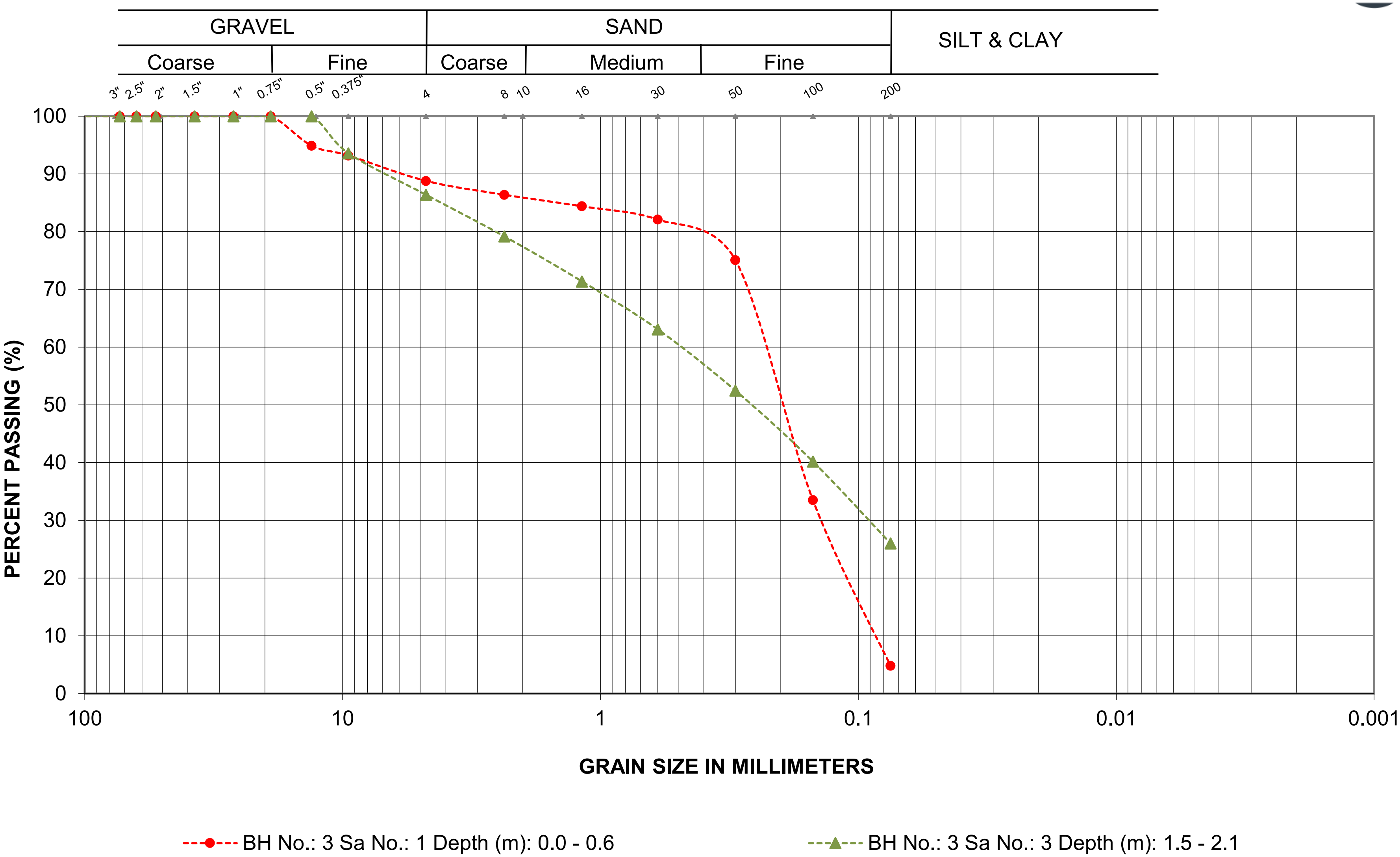
PROJECT: 435 Whitson.
LOCATION: North Bay, ON

Englobe Corp.

FIGURE L-2

Reference No.: 02509305.001
Date: November, 2025

GRAIN SIZE ANALYSIS



SANDS

PROJECT: 435 Whitson.
LOCATION: North Bay, ON

Englobe Corp.

FIGURE L-3

Appendix D
Photo Essay

ENGLOBE

Site Typical – South-east Corner – Looking North-west | Photo: 1

Project No.: 02509305
Project: 435 Whiston St., North Bay, ON

Photos By: Englobe
Date: November 2025

Site Typical – South-east Corner – Looking West | Photo: 2

Project No.: 02509305
Project: 435 Whiston St., North Bay, ON

Photos By: Englobe
Date: November 2025

Appendix E
Seismic and Typical Drawings

2025 - 2020 National Building Code of Canada Seismic Hazard Tool

The NBC 2025 values are subject to change up until the official release of NBC 2025.

This application provides seismic values for the design of buildings in Canada under Part 4 of the National Building Code of Canada (NBC) 2020 and 2025, as prescribed in Article 1.1.3.1. of Division B of the respective NBC editions.

Seismic Hazard Values

User requested values

Code edition	NBC 2025
Site designation X_S	X_C
Latitude (°)	46.301
Longitude (°)	-79.453

Please select one of the tabs below.

NBC 2025 | Additional Values | Plots | API | Background Information

The 5%-damped spectral acceleration ($S_a(T,X)$, where T is the period, in s, and X is the site designation) and peak ground acceleration (PGA(X)) values are given in units of acceleration due to gravity (g, 9.81 m/s^2). Peak ground velocity (PGV(X)) values are given in m/s. Probability is expressed in terms of percent exceedance in 50 years. Further information on the calculation of seismic hazard is provided under the *Background Information* tab.

The 2%-in-50-year seismic hazard values are provided in accordance with Article 4.1.8.4. of the NBC 2025. The 5%- and 10%-in-50-year values are provided for additional performance checks in accordance with Article 4.1.8.23. of the NBC 2025.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2025.

NBC 2025 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_C)$	$S_a(0.5, X_C)$	$S_a(1.0, X_C)$	$S_a(2.0, X_C)$	$S_a(5.0, X_C)$	$S_a(10.0, X_C)$	PGA(X_C)	PGV(X_C)
0.381	0.239	0.129	0.0601	0.0158	0.0053	0.198	0.16

The log-log interpolated 2%/50 year $S_a(4.0, X_C)$ value is : **0.0219**

Next Generation Geophysics

SIMCOE GEOSCIENCE
www.SimcoeGeoscience.com

GEOPHYSICAL REPORT

ENGLOBE

GEOPHYSICAL TESTING FOR SEISMIC SITE CLASSIFICATION

915 McIntyre Street East, North Bay, ON

April 21, 2026 **Project # SGL-#26545**

GENERAL OVERVIEW

Simcoe Geoscience Limited (herein referred to as Simcoe) was requested by Englobe to conduct a seismic site class testing at 915 McIntyre Street east, North Bay, Ontario. An MASW/MAM and an HVSR test were executed to obtain shear wave velocity values from surface. The location is shown in Figure 1.

The survey was carried out on April 14th, 2026. The survey was conducted in the front of the building, In the parking lot. The cultural noise levels from vehicular traffic were considered acceptable. The topography along the line was relatively flat, with approximately 0.5 m of elevation change at the north end of the profile. The ground surface was covered with gravel. A boulder or exposed rock could be seen near the center of the line.

One MASW Sounding was acquired using a single string of 24 geophones with a standard receiver spacing of 3 meters and one HVSR test conducted simultaneously. Seven (7) active shots and twenty (20) passive records were measured and recorded for this sounding, and a 30-minute recording time for the HVSR test. The passive records were collected with a longer sampling rate and longer total recording length. The active records are predominantly intended for modeling the upper 20 meters while the passive records tend to be lower frequency components that are suited for modeling in the 15-to-40-meter range.

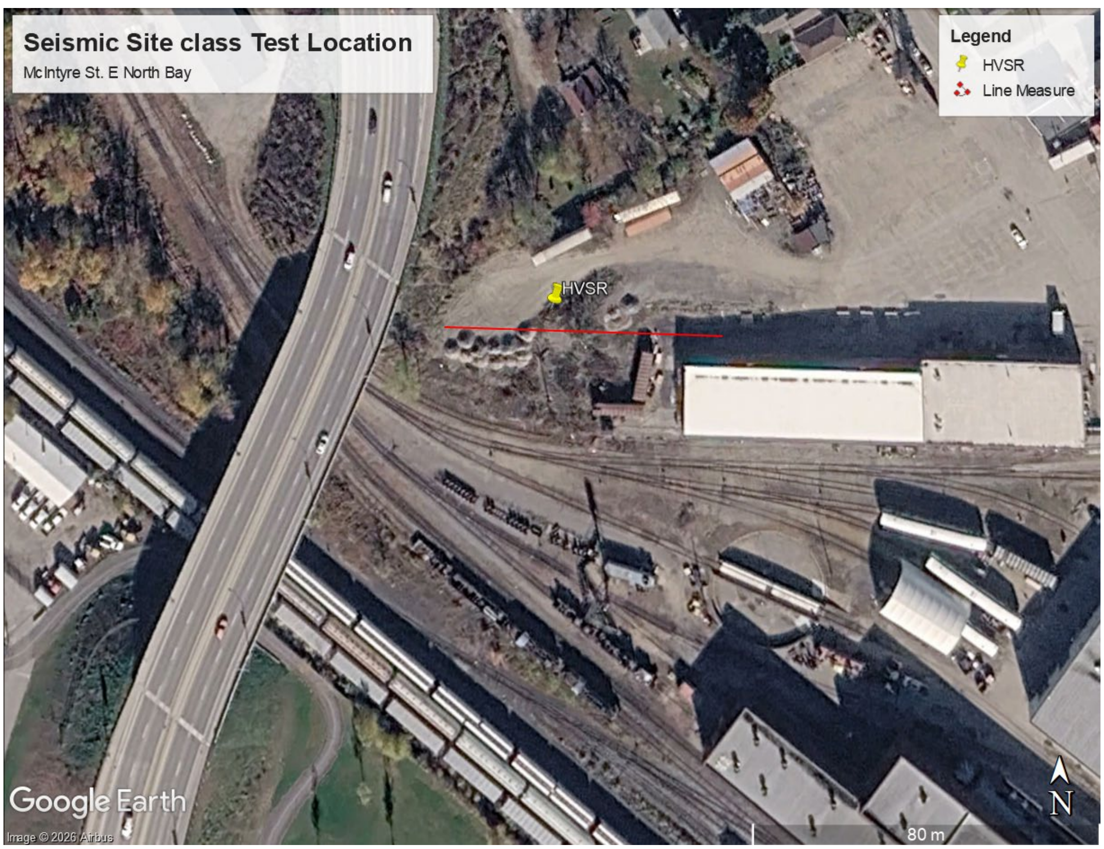


Figure 1: MASW/MAM Sounding and HVSR Locations

FIELD PROCEDURE

The field setup of a MASW survey involves the layout 24 geophones in a linear array. There may be other array designs depending on the site and aim of the survey. Data acquisition involves generating an acoustic wave with a sledgehammer (commonly there are several other sources possible) and digitally recording the rolling surface waves from the moment of source impact for a full second as it passes the entire array of sensors (geophones).

For this study, data was collected with ABEM Terraloc Pro 2™ seismograph - 24 channels and 4.5 Hz geophones. A sledgehammer was used as the primary energy source with traces being recorded at 7 locations: one shot at centre, two shots between the first and last geophones and two shots at 6m and 20m offset beyond the ends of the line. The shot at the centre of the spread is used as a refraction reference for the operator and in processing to see the compression waves rather than shear waves. Figure 2 shows typical field setup for 3-meter receiver interval. Note that the shot locations are different in Figure 2 than in the executed survey. It is common to adjust shot locations depending on a few factors such as rock depth, field conditions (like noise) and intended requirements for the survey.

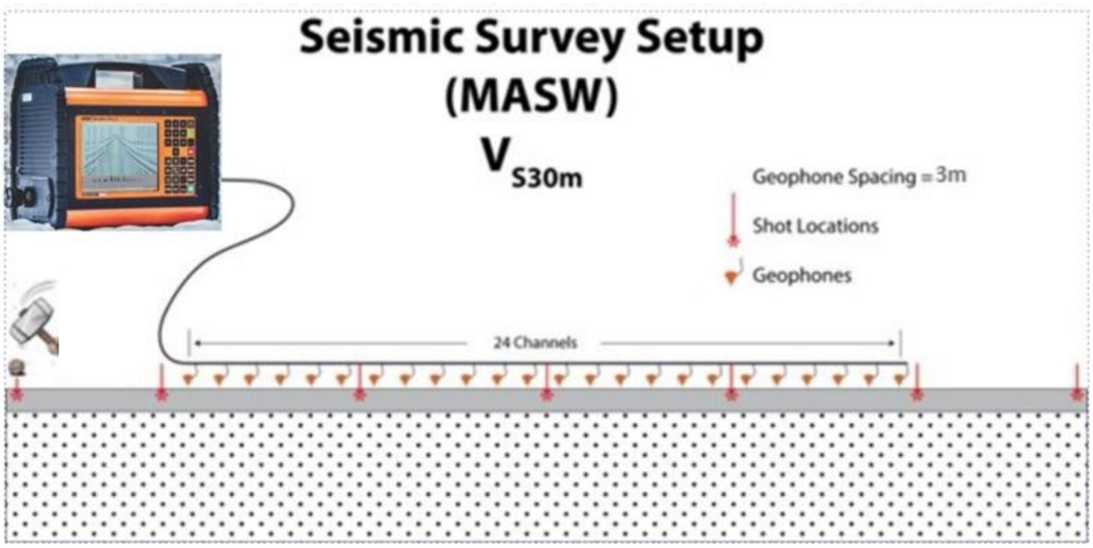


Figure 2: MASW 3-meter Spacing Field Setup, Geophones (orange), Shot Locations (red)

The passive survey (MAM) used the same geophone array set up as for the MASW survey. Unlike the MASW survey, the MAM method is considered a "passive source" method. There is no time break, and the motions recorded are from ambient energy generated by cultural noise such as traffic, wind, wave motion, etc. Data collection for the passive method involved recording approximately 10 minutes of background "noise" for this sounding.

The records generated by the MAM method contain lower frequency data, thus increasing the data modelling to greater depths. Typically, the MAM results help clarify the MASW results for depths greater than 20 m; however, the direction of noise propagation relative to the spread orientation can influence the results. In this survey, the cultural noise direction was parallel to the entire spread which is excellent for data collection.

An additional test was conducted near the centre of the MASW test called HVSR (Horizontal to Vertical Spectral Ratio). This uses a device called a Tromino™ seismograph with three component (three orientations) sensors. The principal objective of the HVSR method is to obtain the fundamental natural frequency of a site, which is often related to the depth of the bedrock or a significant velocity contrast layer. This method collects passive data for 30 minutes at an individual location. The results for this survey were omitted, as the site conditions were not conducive to reliable interpretation.

DATA PROCESSING AND INTERPRETATION

The MASW data was processed and interpreted using SeisImager™ Surface Wave Analysis to generate a 1-D (depth) shear-wave velocity (Vs) profile. The active and passive data were post-processed, and individual dispersion curves were generated and stacked to generate one average dispersion image of the highest signal-to-noise (S/N) ratio. Two separate dispersion images were generated, i.e., active, and passive records.

The passive image was prepared by stacking all individual dispersion images processed from twenty (20) passive field records (30 seconds each). This indicates surface wave energy accumulation at relatively lower frequencies (e.g., ≤ 10 Hz) where the active image significantly lacks any meaningful energy trend.

Finally, both active and passive dispersion images were combined to generate one combined dispersion curve that has the highest resolution and the broadest bandwidth. In the overall dispersion trend, extracting the fundamental-mode dispersion curve (M0), which then indicates that the modal interpretation of M0 is more confident in the combined image. This also has the final 1-D velocity (Vs) profile which will have an increased confidence level at deeper depths (e.g., ≥ 20 m) because of the lower frequencies (e.g., ≤ 10 Hz) thus can then be extracted for the M0 curve. The M0 curve (Figure 3) was then used to generate the final 1-D shear-wave velocity (Vs) profile through the subsequent inversion process. The smoothing of the curve helped to minimize the noise of the data, which could produce extra layers in the 1D results.

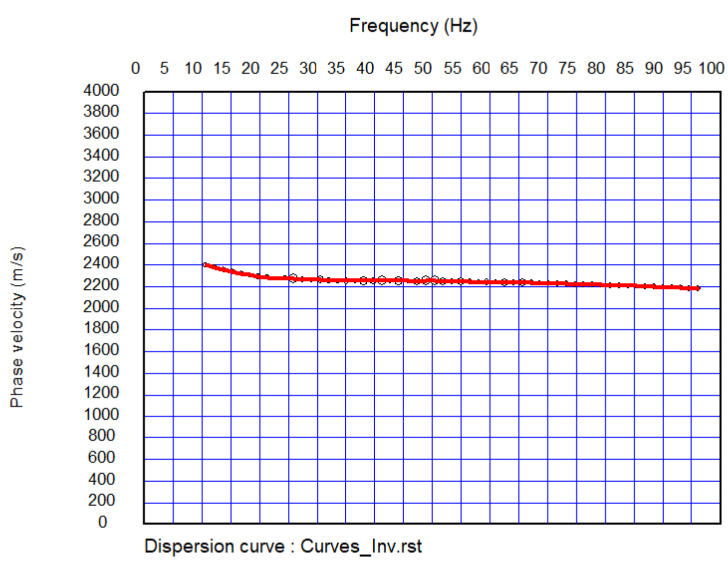


Figure 3: Combined Active & Passive Dispersion Curves

RESULTS

The Rayleigh dispersion curves obtained from the active and passive datasets were of good quality. There was no significant overwhelming cultural noise at this site to disrupt the active records. There is a process called "stacking" which combines records after hitting the same location several times. This helps to maximize the signal to noise ratio.

The inverted model (see Figure 5) suggests that the bedrock is very shallow, within 1 m from surface.

In areas of shallow bedrock where fundamental-mode MASW dispersion curves are not well resolved, independently measured refracted P-wave velocities can be used to constrain shear-wave velocity estimates. Using a conservative Poisson's ratio, the measured P-wave velocities of 5200-5600 m/s correspond to theoretical S-wave velocities on the order of 2500-3000 m/s. A more conservative range of 2000-2500 m/s was adopted to account for potential near-surface fracturing and reduced shear stiffness.

The HVSR survey results are not presented in this report. In cases of very shallow bedrock, the HVSR response is often inconclusive, as the resonance frequencies do not have sufficient depth contrast or time to develop and merge into a clear peak.

The available boreholes confirmed the shallow rock at this site. Boreholes No.6 and 7 show the bedrock on surface.

The available boreholes confirm shallow bedrock at the site. Boreholes N5 and N7 indicate that the bedrock is at surface.

Option #1: Slab-on-Grade Design, 101 m asl

Profile	Depth (m)	Vs30 (m/s)	Seismic Site Class
MASW/MAM	0 to 30	1939.5	A*

* NBC 2020 Commentary "J" requirements

Option #1: Slab-on-Bedrock Design, 100.5 m asl

Profile	Depth (m)	Vs30 (m/s)	Seismic Site Class
MASW/MAM	0.5 to 30.5	2225	A*

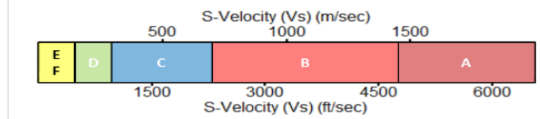
* NBC 2020 Commentary "J" requirements

This site can be classified as **Seismic Site Class A ("Hard Rock")** according to the seismic site classification codes adopted by National Building Code of Canada 2020, Ontario Building Code 2024 and the International Building Code (IBC), see Figure 4.

If we apply the NBCC 2020 and the OBC 2024 codes, there is Table 4.1.8.4.A *Exceptions for Site Designation Using Vs30 Calculated from in Situ Measurements*. X_v = 2225.

The use of site class "A" is conditional on the requirements of Commentary "J" sentence 100, specifically, *"Site Classes A and B, are not to be used if there is more than 3 m of soil between the rock surface and the bottom of* *the spread footing or mat foundation, even if the computed average shear wave velocity is greater than 760m/s".*

Seismic Site Classification ($Vs^{30\text{-}m}$ or $Vs^{100\text{-}ft}$)



NBCC* Seismic site classification based on shear-velocity (Vs) ranges.

Site Class	S-Velocity (Vs) (ft/sec)	S-Velocity (Vs) (m/sec)
A (Hard Rock)	> 5,000	**> 1500**
B (Rock)	2,500 – 5000	**760 – 1500**
C (Very Dense Soil and Soft Rock)	1,200 – 2,500	**360 – 760**
D (Stiff Soil)	600 – 1,200	**180 – 360**
E (Soft Clay Soil)	< 600	**< 180**
F (Soils Requiring Add'l Response)	< 600, and meeting some additional conditions.	**< 180, and meeting some additional conditions.**

*** National Building Code of Canada**

Figure 4: Summary of Site Class based upon Shear-wave velocities

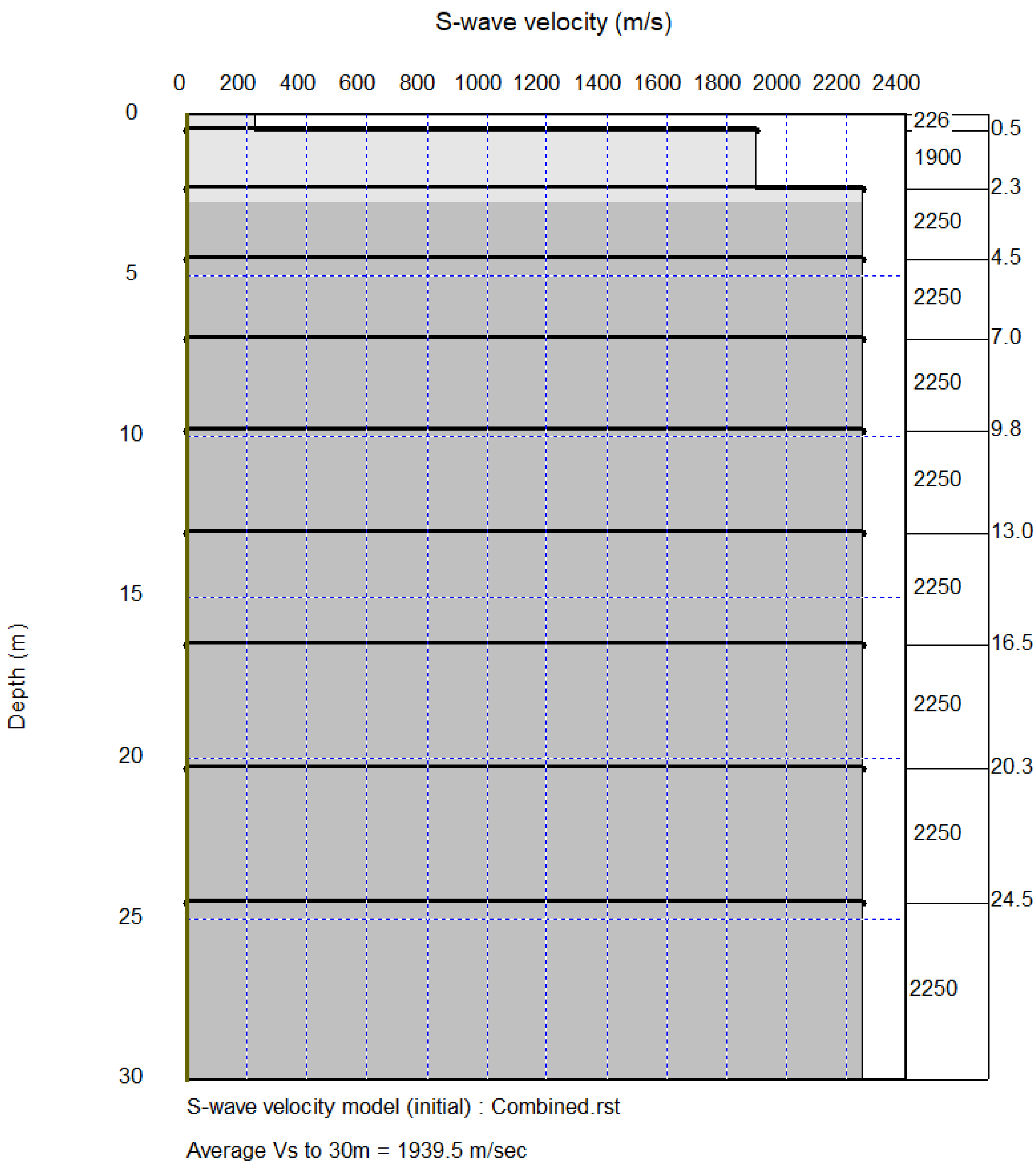


Figure 5: MASW/MAM Sounding results

Figure 5 is the model from the MASW/MAM soundings. The strength of the MASW is its ability to measure soil strength.

CONCLUSIONS

The data quality was good and the dispersion curves were considered acceptable given the site conditions.

The Vs30 from surface to 30 meters is 1939.5 m/s. When considering the building directly on the bedrock the Vs30 would be 2225 m/s.

The seismic site class is "A". The estimated bedrock depth is within a meter from surface at this site.

It is important to note that data analysis and seismic site classification described in this report is based on seismic methods only. The results of MASW sounding can be superseded by other geotechnical information such as the presence of sensitive and/or liquefiable soils, more than 3 meters of soft clays, high moisture content, etc. It is important to consider other geotechnical information prior to further investigations on site. For more details about seismic site classification, the reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2020 Edition.

We are committed to providing the next-generation ground and marine geophysical technologies and expertise to apply to all forms of engineering applications.

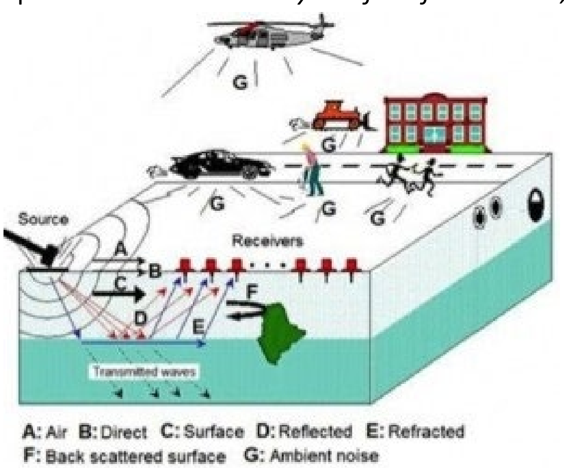
Respectfully submitted,

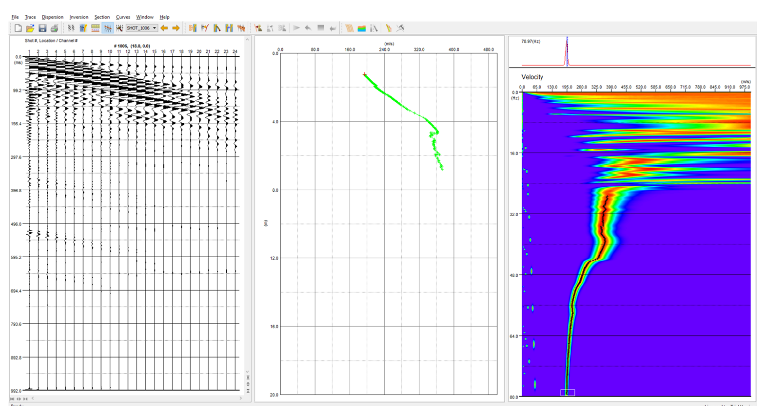
Lhoucin Taghya, P. Geo.
Senior Geophysicist
Simcoe Geoscience Limited

PROFESSIONAL GEOSCIENTIST
LHOUCIN TAGHYA (LT)
PRACTISING MEMBER
2872
ONTARIO

MASW METHOD

First introduced in GEOPHYSICS (1999), the Multichannel Analysis of Surface Waves (MASW) method is one of the seismic survey methods evaluating the elastic condition (stiffness) of the ground for geotechnical engineering purposes. MASW first measures seismic surface waves generated from various types of seismic sources—such as sledgehammer—analyzes the propagation velocities of those surface waves, and then finally deduces shear-wave velocity (Vs) variations below the surveyed area that is most responsible for the analyzed propagation velocity pattern of surface waves. Shear-wave velocity (Vs) is one of the elastic constants and closely related to young's modulus. Under most circumstances, Vs is a direct indicator of ground strength (stiffness) and therefore commonly used to derive load-bearing capacity. After a series of processing and modeling procedures, final Vs information is provided in 1D, 2D and 3D formats. Figures below outline the basic operating procedure for the MASW method and an example image of a typical MASW record and resulting 1D Vs model. The shear-wave depth profile is the average of the bulk area within the middle third of the geophone spread. The nominal maximum depth of penetration is half of the maximum seismic array length, which in practice is often influenced by the geology. A more detailed description of the method can be found in the paper *Multi-channel Analysis of Surface Waves,* Park, C.B., Miller, R.D. and Xia, J. Geophysics, Vol. 64, No. 3 (May-June 1999); P. 800–808.





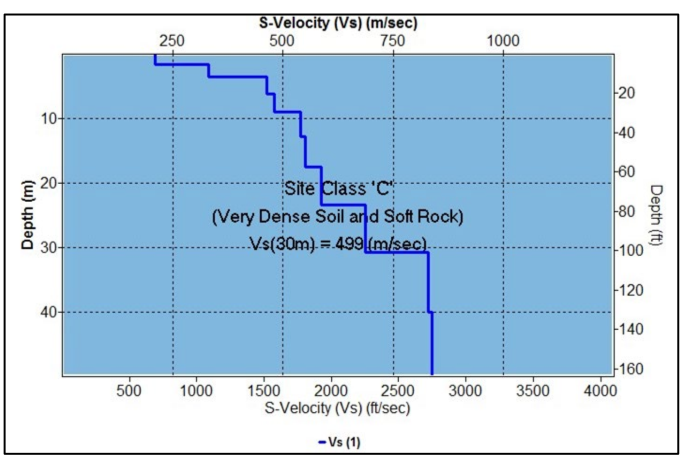
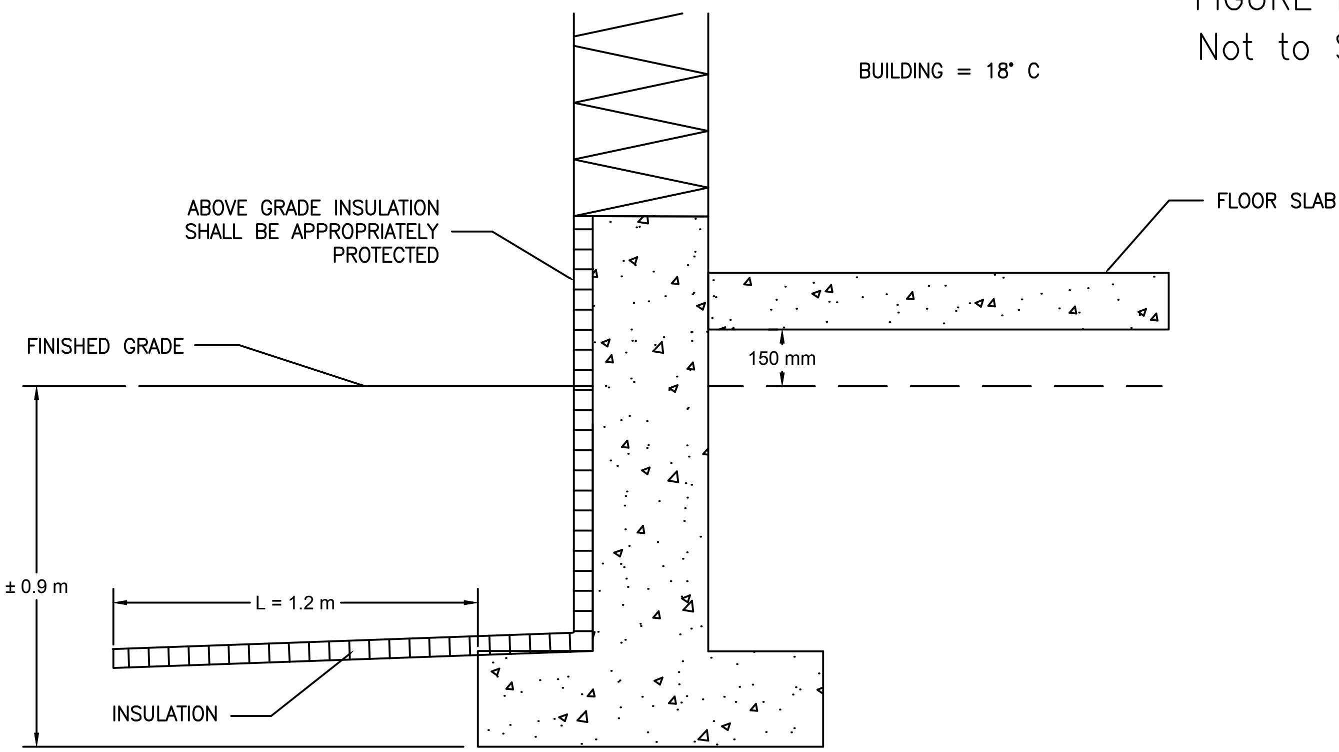


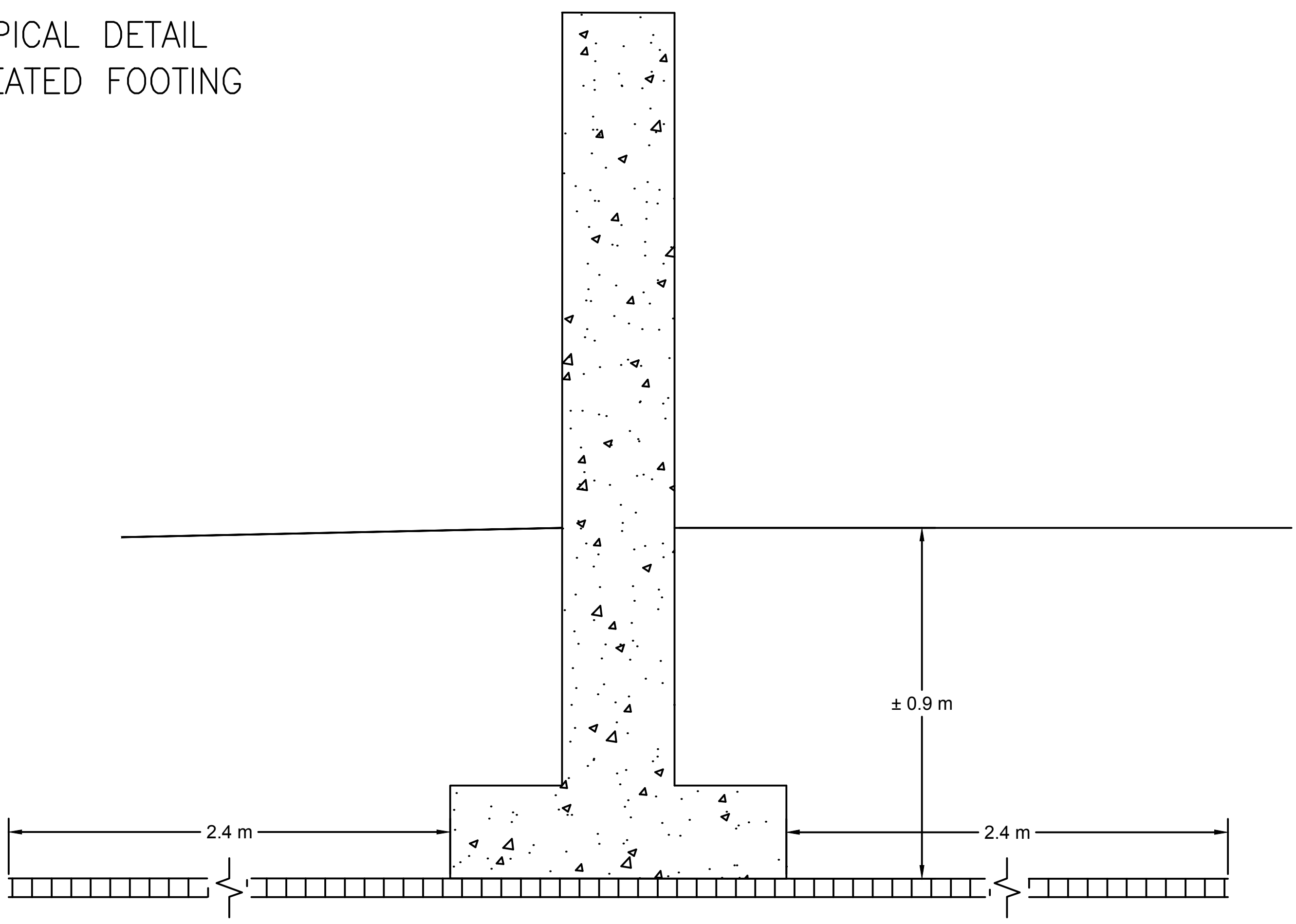
FIGURE No. B

Not to Scale

TYPICAL DETAIL
HEATED FOOTING



TYPICAL DETAIL
UNHEATED FOOTING



Appendix F

Rock Probe Memo

April 22, 2026

Ontario Northland Transportation Commission
Rock Probe Program - 435 Whitson St.
555 Oak Street East
North Bay, ON Canada
P1B 8E3

Attention: Sharon Zwaan, P.Eng. – Project Manager Facilities

Subject: **Technical Memo – Rock Probe Program**
Ontario Northland Transportation Commission
Englobe Reference: 02509305.002

Englobe Corp. (Englobe) was retained by Ontario Northland Transportation Commission (ONTC; the 'Client') to oversee rock probe drilling work at the 435 Whitson Street project in North Bay, Ontario (the Site), as requested by the Client.

A geotechnical investigation was undertaken by Englobe at the site in December of 2025 and must be read in conjunction with this technical memo including the limitations:

Geotechnical Investigation for ONTC Office Building at 435 Whitson St. North Bay, Ontario; Ontario Northland; December 12, 2025; Englobe Reference 02509305.001

The rock probe program was intended to provide the design team with the general depth to bedrock throughout the area of planned development, should the decision be made to take all foundations down to bear directly on bedrock. The rock probe program was conducted on April 13, 2026. The program included the advancement of twelve (12) rock probes. The locations of the rock probes are shown on the Rock Probe Location Plan in Appendix A.

The rock probes were advanced through the use of a pneumatic rock drill operated by Bayroc Driling Blasting and Excavating, and bedrock surface was provided by the drill operator based on the response of the drill and their experience. Rock probes were advanced unsampled through overburden to determine refusal on presumed bedrock, to depths ranging from 0.3 m below ground surface (bgs) to 4.3 m bgs, with an average of 1.4 m. The ground elevations and refusal details are provided in the following table and in the Rock Probe Logs in Appendix B. Photographs collected during field work are provided in Appendix C.

Table 1: Rock Probes - Elevations and Refusal Depths

Rock Probe ID	Estimated Ground Elevation (m)	Refusal Depth (m)	Estimated Refusal Elevation (m)
Rock Probe No. 1	202.1	2.8	199.3
Rock Probe No. 2	202.1	1.0	201.1
Rock Probe No. 3	202.3	0.8	201.5
Rock Probe No. 4	202.2	0.7	201.5
Rock Probe No. 5	202.2	2.2	200.0

Rock Probe ID	Estimated Ground Elevation (m)	Refusal Depth (m)	Estimated Refusal Elevation (m)
Rock Probe No. 6	202.3	1.3	201.0
Rock Probe No. 7	202.5	0.3	202.2
Rock Probe No. 8	202.3	0.7	201.6
Rock Probe No. 9	202.1	4.3	197.8
Rock Probe No. 10	202.5	1.7	200.8
Rock Probe No. 11	202.6	0.6	202.0
Rock Probe No. 12	202.5	0.3	202.2

As can be seen from the above table, the bedrock is estimated to be relatively shallow throughout the area of planned development but quite erratic in nature. It is again noted that bedrock surfaces in the area are known to be erratic in nature and vary substantially in elevation over short horizontal distances.

We thank the ONTC for the opportunity to complete this work and trust that we have satisfied your present requirements. Should you require additional information, please do not hesitate to contact the undersigned.

Yours very truly,

Englobe Corp.

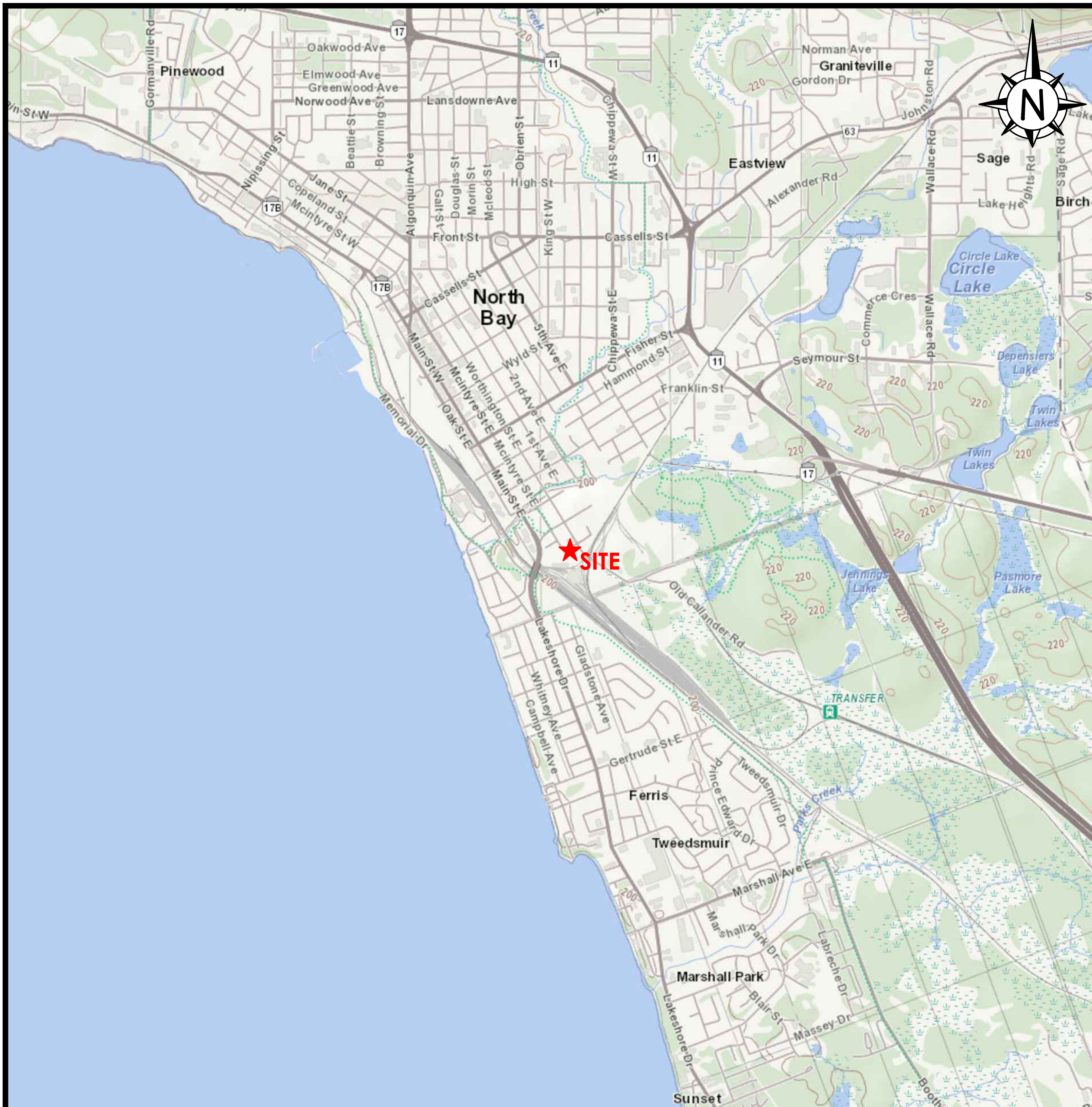
Duncan Mitchell
Design Technician
Materials and Geotechnical, NE/NW ON

J. R. Berghamer, P. Eng., QPESA
Director of Operations
Materials and Geotechnical, NE/NW ON

APPENDICES

Appendix A	Drawings
Appendix B	Rock Probe Logs
Appendix C	Photo Essay

Appendix A
Drawings



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No.	Version	Date	By	Verif	Appr.
00		2025/11/07	DM	JRB	JRB

ONTC

Geotechnical Investigation
435 Whitson St.
North Bay, ON

Key Plan (Macro)

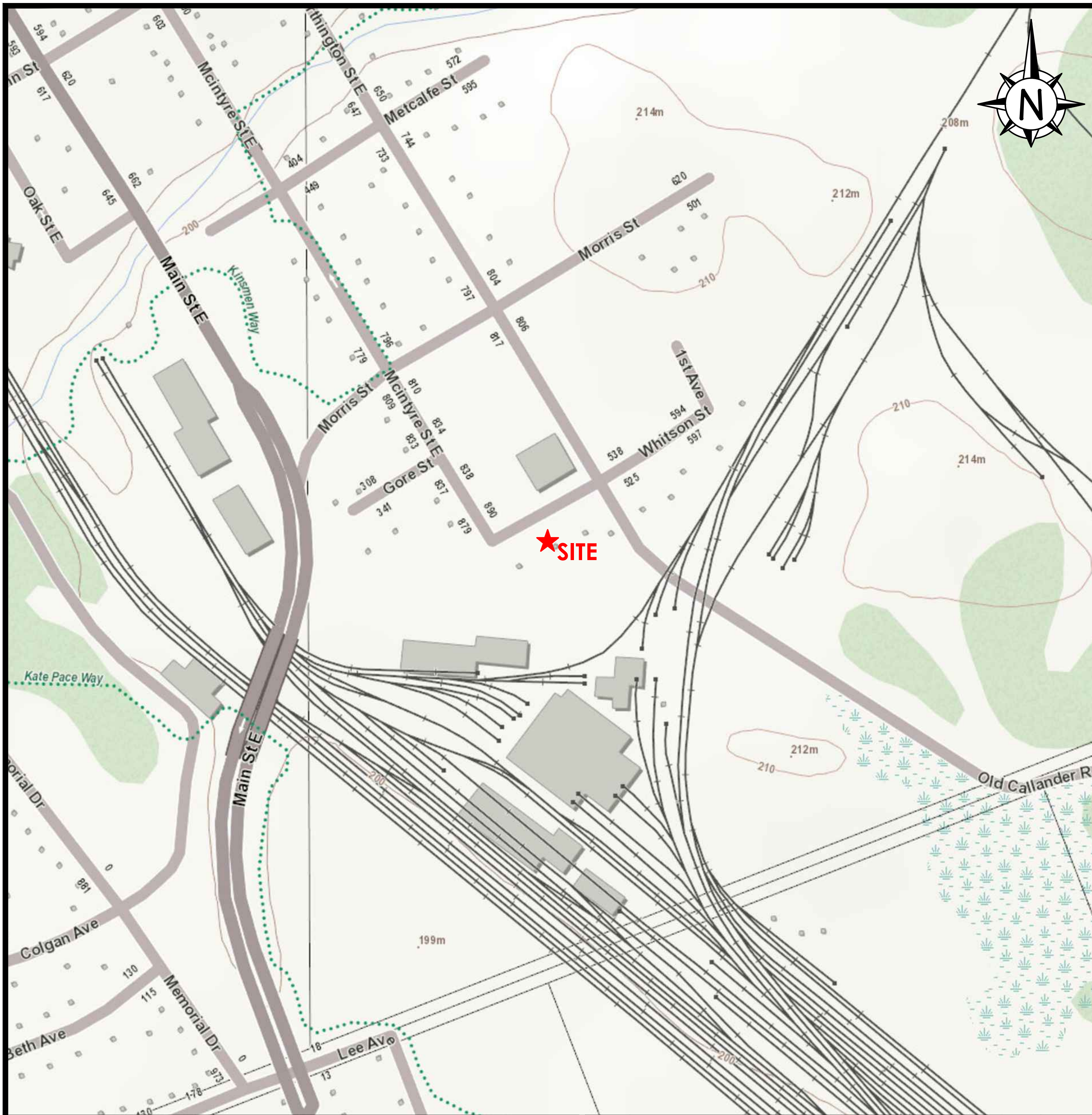
ENGLOBE

2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical		Prepare by: DM	Verify by: JRB
Scale: Not To Scale		Draw by: DM	Approval by: JRB
Date: 2025/11/07		Drawing no: 1a	
Page setup: Macro	Paper size: 8.50 X 11.00 in.	Register no.: ----------	

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
JRB	02509305	003	GE	-	--	--	----	00

D:\Backup\Transfer\Work\02509305.003 - 435 Whiston Probes\02509305.003 - 435 Whiston - Rock Probe Location Plan, 435 Whitson.dwg



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00		2025/11/07	DM	JRB	JRB

ONTC

Geotechnical Investigation
435 Whitson St.
North Bay, ON

Key Plan (Micro)

ENGLOBE

2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical	Prepare by: DM	Verify by: JRB	
Scale: Not To Scale	Draw by: DM	Approval by: JRB	
Date: 2025/11/07	Drawing no: 1b		
Page setup: Micro; Paper size: 8.50 X 11.00 in.	Register no.: ----------		

Man.	Project	Otp	Project	Phase	Electronic ref.			Rev.
JRB	02509305	003	GE	-	--	--	----	00

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No.	Version	Date	By	Verif	Appr.
00		2026/04/22	DM	JRB	JRB

ONTC

Geotechnical Investigation
435 Whitson St.
North Bay, ON

Rock Probe Location Plan

ENGLOBE
2–120 Progress Court
North Bay, Ontario, P1A 0C2
705-476-2550

Discipline: Geotechnical		Prepare by: DM	Verify by: JRB
Scale: Not To Scale		Draw by: DM	Approval by: JRB
Date: 2026/04/22		Drawing no: 2	
Page setup: BH Plan	Paper size: 8.50 X 11.00 in.	Register no.: ----------	

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
JRB	02509305	003	GE	-	--	--	----	00

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Appendix B
Rock Probe Logs

LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

1. ABBREVIATIONS

AS Auger Sample
CS Chunk Sample
DS Denison type sample
FS Foil Sample
NFP No Further Progress
PH Sampler advanced by hydraulic pressure
PM Sampler advanced by manual pressure
RC Rock core with size & percentage of recovery
SS Split Spoon
ST Slotted Tube
TO Thin-walled, open
TP Thin-walled, piston
WS Wash Sample

2. PENETRATION RESISTANCE/"N"

Dynamic Cone Penetration Test (DCPT):

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as —•—•—•—•

Standard Penetration Test (SPT) or "N" Values

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	*Compactness Condition*
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

3. SOIL DESCRIPTION (Cont'd)

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	*Consistency*
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

c) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

+ 3.2 - Field Vane test in borehole. The number denotes the sensitivity to remoulding.

D - Laboratory Vane Test

·· - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

5. LABORATORY TESTS

P Standard Proctor Test
A Atterberg Limit Test
GS Grain Size Analysis
H Hydrometer Analysis
C Consolidation

SAMPLE DESCRIPTION NOTES:

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.

2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.

3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.

4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

ENCLOSURE NO.:2 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 01

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
202.1	Ground Surface										
0.0	Start Rock Probe						202				
							201				
							200				
199.3 2.8	Probe Refusal - Presumed Bedrock End of Rock Probe										

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

Englobe Corp.

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ENCLOSURE NO.:3 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 02

englobe

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

ELEV DEPTH	SOIL PROFILE: DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
202.1	Ground Surface										
0.0	Start Rock Probe						202				
201.1 1.0	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

	Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)		-	-
2)		-	-
3)		-	-

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METRIC

RECORD OF ROCK PROBE NO. 03

ENGLOBE

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
202.3	Ground Surface										
0.0	Start Rock Probe						202				
201.5 0.8	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

O $^{3\%}$ STRAIN AT FAILURE

WATER LEVEL RECORDS		
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

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METRIC

RECORD OF ROCK PROBE NO. 04

ENGLOBE

REFERENCE 02509305.003	DATUM Geodetic	LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2	ORIGINATED BY JS
PROJECT 435 Whitson St., Rock Probes		BOREHOLE TYPE Pneumatic Rock Drill	COMPILED BY DM
CLIENT ONTC		DATE (Started) 13 April 2026 / DATE (Completed) 13 April 2026 / TIME (Completed)	CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
202.2	Ground Surface										
0.0	Start Rock Probe						202				
201.5 0.7	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

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ENCLOSURE NO.:6 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 05

ENGLOBE

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
202.2	Ground Surface										
0.0	Start Rock Probe						202				
							201				
200.0 2.2	Probe Refusal - Presumed Bedrock End of Rock Probe						200				

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+³, ×³ : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

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ENCLOSURE NO.:7 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 06

ENGLOBE

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
202.3	Ground Surface										
0.0	Start Rock Probe						202				
201.0 / 1.3	Probe Refusal - Presumed Bedrock End of Rock Probe						201				

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

+3, ×3 : Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

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ENCLOSURE NO.:8 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 07

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
202.5	Ground Surface										
0.0	Start Rock Probe										
202.2 0.3	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS		
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

ENCLOSURE NO.:9 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 08

ENGLOBE

REFERENCE	02509305.003	DATUM	Geodetic	LOCATION	See Rock Probe Location Plan; Appendix B, Drawing No. 2
ORIGINATED BY	JS				
PROJECT	435 Whitson St., Rock Probes	BOREHOLE TYPE	Pneumatic Rock Drill	COMPILED BY	DM
CLIENT	ONTC	DATE (Started)	13 April 2026	TIME	
		DATE (Completed)	13 April 2026	(Completed)	
CHECKED BY	JRB				

ELEV DEPTH	SOIL PROFILE: DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
202.3	Ground Surface										
0.0	Start Rock Probe						202				
201.6 0.7	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3$, $\times^3$: Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ $^{3\%}$ STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

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ENCLOSURE NO.:10 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 09

REFERENCE 02509305.003	DATUM Geodetic	LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2	ORIGINATED BY JS
PROJECT 435 Whitson St., Rock Probes		BOREHOLE TYPE Pneumatic Rock Drill	COMPILED BY DM
CLIENT ONTC		DATE (Started) 13 April 2026 / DATE (Completed) 13 April 2026 / TIME (Completed)	CHECKED BY JRB

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	NUMBER	TYPE	"N" VALUES						
202.1	Ground Surface										
0.0	Start Rock Probe						202				
							201				
							200				
							199				
							198				
197.8 4.3	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

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ENCLOSURE NO.:11 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 10

ENGLOBE

REFERENCE 02509305.003 DATUM Geodetic LOCATION See Rock Probe Location Plan; Appendix B, Drawing No. 2 ORIGINATED BY JS

PROJECT 435 Whitson St., Rock Probes BOREHOLE TYPE Pneumatic Rock Drill COMPILED BY DM

CLIENT ONTC DATE (Started) 13 April 2026 DATE (Completed) 13 April 2026 TIME (Completed) CHECKED BY JRB

ELEV DEPTH	SOIL PROFILE DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
202.5	Ground Surface										
0.0	Start Rock Probe						202				
							201				
200.8 1.7	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

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ENCLOSURE NO.:12 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 11

ENGLOBE

REFERENCE	02509305.003	DATUM	Geodetic	LOCATION	See Rock Probe Location Plan; Appendix B, Drawing No. 2
ORIGINATED BY	JS				
PROJECT	435 Whitson St., Rock Probes	BOREHOLE TYPE	Pneumatic Rock Drill	COMPILED BY	DM
CLIENT	ONTC	DATE (Started)	13 April 2026	DATE (Completed)	13 April 2026
TIME (Completed)		CHECKED BY	JRB		

ELEV DEPTH	SOIL PROFILE – DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L – WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
202.6	Ground Surface										
0.0	Start Rock Probe										
202.0 0.6	Probe Refusal - Presumed Bedrock End of Rock Probe						202				

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 100 kPa

O $^{3\%}$ STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

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ENCLOSURE NO.:13 (Pg. 1 of 1)

METRIC

RECORD OF ROCK PROBE NO. 12

ENGLOBE

REFERENCE	02509305.003	DATUM	Geodetic	LOCATION	See Rock Probe Location Plan; Appendix B, Drawing No. 2
ORIGINATED BY	JS				
PROJECT	435 Whitson St., Rock Probes	BOREHOLE TYPE	Pneumatic Rock Drill	COMPILED BY	DM
CLIENT	ONTC	DATE (Started)	13 April 2026	DATE (Completed)	13 April 2026
TIME (Completed)		CHECKED BY	JRB		

ELEV DEPTH	SOIL PROFILE – DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
202.5	Ground Surface										
0.0	Start Rock Probe										
202.2 0.3	Probe Refusal - Presumed Bedrock End of Rock Probe										

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 02509305.003 - ROCK PROBE LOGS, 435 WHISTON.GPJ MEL-GEO.GDT 1/5/26

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Appendix C
Photo Essay

ENGLOBE

Site Typical – Shannondale – Facing South | Photo: 1

Project No.: 02509305.002
Project: 435 Whitson Street, North Bay, ON

Photos By: Englobe
Date: April 2026

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Site Typical – Facing West	Photo: 2

Project No.: 02509305.002 Project: 435 Whitson Street, North Bay, ON	Photos By: Englobe Date: April 2026

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