Proposed Housing Development
243 River Road, Sault Ste Marie, ON

Geotechnical Investigation

Ontario Aboriginal Housing Services
Canada Mortgage and Housing Corporation
Final GI Report
Proposed Housing Development

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243 River Road
Sault Ste Marie, ON

Prepared by:

DMcKay

Diana McKay C.E.T.
Technologist
Geotechnical and Materials (NE/NW Ontario)

Richard Twiss, M.Sc., E.I.T.
Engineer-in-Training
Geotechnical & Materials (NE/NW-ON)

Reviewed by:

LICENSED PROFESSIONAL ENGINEER
J. R. BERGHAMER
2024-04-08
PROVINCE OF ONTARIO

Jake Berghamer, P. Eng
Director of Operations
Geotechnical and Materials (NE/NW Ontario)

Bernardo Villegas, M.Sc.
Geotechnical Specialist
Geotechnical and Materials (NE/NW Ontario)

Client:

Ontario Aboriginal Housing Services	Paula Benford, Development Coordinator

Production Team:

Englobe Corp.

Project Manager	Richard Twiss, E.I.T. M.Sc.
Drafting	Diana McKay, C.E.T.
Project Engineer	Jake Berghamer, P. Eng
Project Specialist	Bernardo Villegas, M.Sc.

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1 Introduction

As requested by Ontario Aboriginal Housing Services (OAHS, the Client) Englobe Corp. (Englobe) has carried out the geotechnical investigation to assess the soil conditions at the location of a proposed new residential development located at 243 River Road in Sault Ste Marie, Ontario (see Key Plans, Drawings No. 1a and 1b, Appendix A). We have completed the field and laboratory testing programs and submit the factual results in this report along with our comments and recommendations.

It is understood that the new housing development will consist of a two-storey building without a basement, with thirty (30) bachelor units, along with a driveway and parking area. The concept drawing provided by the client is included in Appendix A (noting that the building concept has been changed to two stories versus three).

The purpose of the geotechnical investigation was to ascertain the subsurface and groundwater conditions at the location of the proposed new buildings to provide geotechnical recommendations for development of the buildings and the site.

1.1 Site Conditions

The area proposed for the new development is located on the east side of River Road, west of St. Mary's River in Sault Ste Marie, Ontario, next to existing residential units and commercial land. The site is located within a glaciolacustrine plain with primarily sands and silty clays.

Underground utility service clearances were undertaken in advance of the investigation. Utilities were not identified in the areas of the borehole investigations.

See Photo Essay Appendix D for existing site conditions.

2 Fieldwork

The fieldwork for this geotechnical investigation was carried out from January 30th to 31st, 2024. The fieldwork consisted of seven (7) sampled boreholes (Boreholes (BH) Nos. 1 to 7).

The locations of the boreholes are shown on the Borehole Location Plan, Drawing No. 2 in Appendix A.

Borehole Nos. 1 to 7 were advanced with a track mounted Geoprobe drill rig operated by North Drilling and equipped with continuous flight hollow stem augers and direct push casings. The field work was under the full-time direction of an experienced member of our engineering field staff who was responsible for underground service locates, 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. Where encountered and possible, the in-situ shear strength of fine-grained deposits was measured using a calibrated torque meter and vane. Dynamic Cone Penetration Test (DCPT) was undertaken beside Boreholes BH Nos. 2, 5, and 7.

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

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 location of each borehole was obtained using a handheld GPS with an accuracy of ±3 m horizontally. Ground surface elevations at the borehole locations were obtained with rod and level survey tied into a temporary local benchmark assigned an elevation of 100.0 m and are provided for reference only. The temporary benchmark is described as the top of the concrete slab at the man door on the east side of the hanger building (as shown in Appendix A, Drawing 2). 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 boring 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, DCPT, 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, DCPT, 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 is defined as the point at which the augers can no longer be practically advanced with the equipment used in this investigation. Refusal, if encountered, to further advance of the augers, DCPT and SPT may have been due to the presence of very dense soils, cobbles/boulders in the underlying soils, or possibly bedrock. Defining the nature of auger refusal with diamond drilling operations was outside the scope of work for this project.

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

3.1 Subsurface Summary Description

3.1.1 Proposed Housing Development

Boreholes (BH) 1 to 5 were put down in the area of the proposed new residential buildings, and BH 6 and 7 in the area of the proposed parking lot. The surface elevation of BHs 1 to 7 were measured at ±101.0, 100.8, 101.9, 101.1, 101.0, 101.3, and 101.6 m, respectively, based on a rod and level survey (as noted, elevations must be confirmed by an OLS).

It is noted that the stratigraphy was observed to be highly variable and interlayered, likely due to the proximity of the site to the river.

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

Borehole ID	Approximate Depth (m, unless otherwise specified)					
	Topsoil	Silt	Silty Clay to to Clayey Silt	Sand, Silty Sand, Sand and Gravel	Sand & Gravel	Silty Sand
BH 1	0.0 - 0.25	0.25 - 1.1	1.1 - 1.6 2.1 - 4.6	1.6 - 2.1 4.6 - 6.1[2]	-	-
BH 2[1]	0.0 - 0.25	-	0.8 - 1.6 2.3 - 3.2 4.3 - 5.2	0.25 - 0.8 1.6 - 2.3 3.2 - 4.3 5.2 - 6.1[2]	-	-
BH 3	0.0 - 0.08	-	3.1 - 3.7 4.4 - 7.0[2]	0.08 - 3.1 3.7 - 4.4	-	-
BH 4	0.0 - 0.15	-	0.8 - 1.0 2.3 - 4.7	0.15 - 0.8 4.7 - 6.1[2]	1.0 - 2.3	-
BH 5[1]	0.0 - 0.1	-	0.8 - 1.5 2.3 - 4.6 7.6 - 9.1[2]	0.1 - 0.8 1.5 - 2.3 4.6 - 7.6	-	-
BH 6	0.0 - 0.15	-	0.8 - 1.5	0.15 - 0.8 1.5 - 2.9[2]	-	-
BH 7[1]	0.0 - 0.05	0.05 - 0.8	3.1 - 3.4	3.4 - 3.8[2]	-	0.8 - 3.1

Note:

(1) Dynamic Cone Penetration Testing (DCPT) performed beside this borehole.

(2) Borehole sampling terminated in this stratum.

3.1.1.1 Topsoil

At the surface in BH 1 to 7, a layer of topsoil was encountered. This layer thickness ranged from 0.05 to 0.25 m. The natural moisture content measured on samples of this layer was in the order of 17 to 34%.

3.1.1.2 Silt

Underlying the topsoil at BH 1 and 7, a deposit of silt, some sand was observed. The investigated thickness of the layer ranged from 0.75 to 0.85 m. Based on SPT ‘N’ value of 5 to 6 blows per 300 mm penetration, the compactness of this deposit was described as “loose”.

3.1.1.3 Silty Clay

Underlying the silt and/or sands in BH 1 to 7, deposits of silty clay, trace sand with interbedded sand seams, was observed. The investigated thickness of these layers ranged from 0.2 to 2.6 m. The natural moisture content measured on samples of these layers ranged from 27% to 64%. The estimated undrained shear strength from field vane tests of this stratum ranged from 48 to 213 kPa indicating a “firm” to “hard” consistency, generally stiff, although it is noted that the sand seams likely impacted the shear vane test results to the upside and the clay is more of a firm consistency. Grain size distribution (hydrometer) was carried out on two (2) samples, and Atterberg Limit Testing was carried out on six (6) samples of these stratums. The results of these tests are provided in Tables 3.2 and 3.3 below and summarized in Appendix C – Laboratory Test Results.

3.1.1.4 Sand, Silty Sand, Sand & Gravel

Underlying the silty clays and silts in BH 1 to 7, deposits of sand, with varying degrees of silt, gravel and clay seams was observed. The investigated thickness of theses layers ranged from 0.4 to 3.0 m. The natural moisture content measured on samples of these layers ranged from 9% to 31%. Based on an SPT ‘N’ values of 0 to 16 blows per 300 mm penetration; the consistency of this deposit was described as “very loose” to “compact”, generally loose. Grain size distribution (sieve) was also carried out on seven (7) samples of these stratums. The results of these tests are provided in Table 3.4 below and also summarized in Appendix C – Laboratory Test Results.

3.1.2 DCPT Results

Sampling was terminated in the sand and silty clay deposits at BH Nos. 2, 5, and 7 and a Dynamic Cone Penetration Test was undertaken from the surface at 1.5 m east, 0.9 m south, and 0.9 m south of the original boreholes, respectively. The terminus depths reached were 13.4 m, 14.9 m, and 13.4 m respectively below ground surface without encountering refusal or any significant increase in resistance.

3.2 Summary of Laboratory Results

The following summarizes the laboratory data results obtained from relevant samples collected during the geotechnical investigation. Samples were obtained during the investigation at the borehole location at the standard depth increments.

The following laboratory tests were carried out to determine the general soil physical characteristics at select borehole location:

- Natural Moisture Content
- Gradation Analysis (sieve and hydrometer)
- Atterberg Limits

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

Table 3-2: Gradation Analysis Results (Sieve and Hydrometer)

Borehole ID	Depth (m)	Material	Gradation			
			Gravel (%)	Sand (%)	Silt (%)	Clay (%)
BH 2	0.8	Clayey Silt	0	5	68	27
BH 2	1.6	Sand	0	87	13	
BH 2	3.2	Sand	0	98	2	
BH 3	0.8	Sand	0	98	2	
BH 3	2.3	Sand	0	98	2	
BH 5	1.5	Sand	0	89	11	
BH 5	4.5	Sand	0	97	3	
BH 6	0.8	Clayey Silt	0	6	67	27
BH 7	0.8	Silty Sand	7	57	36	

Table 3-3 summarizes the Atterberg Limit results obtained from the laboratory testing on the following samples:

Table 3-3: Atterberg Limits Results on Silty Clay

Borehole ID	Depth (m)	Atterberg Limits			
		LL (%)	PL (%)	PI (%)	USCS
BH 2	0.8	34	21	13	CL
BH 2	2.3	55	24	31	CH
BH 2	4.6	58	26	32	CH
BH 3	4.6	49	19	30	CI
BH 5	2.3	48	18	30	CI
BH 5	7.6	39	17	22	CI

3.3 Groundwater Data

Groundwater and cave-in levels in the open boreholes were measured, where possible, during the advance of the individual borings and upon completion. It is noted that there may have been insufficient time for the groundwater levels to stabilize in the boreholes prior to measuring. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix B) and the Table 3-4 below.

Table 3-4: Water Level Measurements

Boring ID	Ground Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)
BH 1	101.0	0.6	100.4
BH 2	100.8	0.6	100.2
BH 3	101.9	1.1	100.8
BH 4	101.1	Not observed Wet soil was indicated at 0.9 mbgs	n/a
BH 5	101.0	Not observed Wet soil was indicated at 0.1 mbgs	n/a
BH 6	101.3	1.1	100.2
BH 7	101.6	1.3	100.3

Groundwater levels will fluctuate seasonally and/or yearly. As such, the groundwater level should 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.

4 Discussion and Recommendations

It is understood that conceptually the new residential development will consist of a two-storey building consisting of thirty (30) bachelor units along with driveway and parking area. The building will not have a basement. Site grade raises above the existing ground elevation are not anticipated. Finished floor elevations were not provided at the time of the report; however, it is expected that finished floor will be close to existing grade. It is anticipated that the slab/floor of the building will be a slab on grade supported on improved ground and engineered fill.

To summarize the borehole findings:

- Below the topsoil, loose sand layers alternating with generally firm silty clay layers were encountered,
- DCPT were advanced up to a depth of 14.9 m below grade without encountering significant resistance,
- Groundwater at the site is relatively shallow, measured at some 0.6 m below grade at the time of the investigation.

4.1 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.

4.2 Foundation Alternatives

As noted, the overburden stratigraphy was observed to be highly variable and interlayered, likely due to the proximity of the site to the river. The overburden strata at this site have relatively low natural bearing resistances with a high groundwater table. Englobe assessed various types of shallow foundations and deep foundation systems relative to the proposed type of structure and the natural overburden, including:

1) Shallow foundations:
    a) Spread footings, and
    b) Strip footings.
2) Deep foundations:
    a) Caissons,
    b) Helical piles,
    c) Friction piles,
    d) Driven Pipe/H-piles, and
    e) Micro-piles.

As noted, for shallow foundations, the bearing resistance of the upper layers of the natural overburden is low, and soil improvement would be required to increase the bearing resistance for shallow foundations. Ground improvement to increase the bearing would likely consist of a combination of geosynthetics and engineered fill, placed below the underside of the foundations. The pad construction requires an oversized excavation, and the dewatering must be undertaken to a depth well below the working subgrade surface. However, to increase the bearing at the depth needed for frost protection would require significant dewatering operations to prevent “boiling” of the subgrade material. A full building footprint dewatering system would likely consist of well points spread evenly around the perimeter of the site. For a dewatering contractor to design the system would require a hydrogeological study and then likely application for a Permit to Take Water. The effect of dewatering within the zone of influence on the surrounding buildings/structures should be also evaluated. The challenges of this approach are significant. However, small footprint excavations, for example for individual spread footings or limiting the length of excavation for strip footings to, for example, 6 m segments or less, may be possible using typical sump hole and pump dewatering techniques, which would avoid the need for a PTTW. It is emphasized that this method would need to be tightly controlled/supervised and requires that the contractor undertakes dewatering operations well in advance of reaching the subgrade level, likely by installing sumps several days prior to excavating. An experienced contractor will be needed for this operation, who provides a written plan in advance of excavating.

From discussions with the client, it is understood that shallow foundations are preferred due to the likely lower cost of shallow foundations compared to deep foundations. However, due to the challenges mentioned above, the new structure may need to be supported on deep foundations. Deep foundations would likely need to be advanced to bedrock which is expected to be some 60 m+ below grade from historical data within the area.

Caisson foundations are typically end bearing and are not recommended for the deep foundation system at this site in consideration of the overburden conditions (i.e., loose sand and silty clay), the potential for sloughing soils, the high groundwater table, and overburden depth. The anticipated cost of caisson foundations likely exceeds other options, and installation challenges, like the need for full casing during caisson advancement, excludes this option.

Typically, friction piles are considered for deep foundation design only when bedrock/practical refusal is located at depths greater than 50 m below grade as is the case at this site (based on historical records published information). However, friction piles are also not recommended for deep foundation system at the site in consideration of practical auger and DCPT refusal were not observed within the typical depth of friction piles. Due to the very loose to compact nature of the overburden soils, a large number (relative to end bearing piles) of friction piles would be needed to develop suitable resistance.

Similarly, helical piles are not considered suitable due to the low resistances observed in the overburden which would likely result in the helical piles having to extend to significant depth to develop sufficient resistance for bearing, and therefore no significant cost savings could be realized.

Micropiles are another deep foundation option, however considering the depth of overburden, this type of deep foundation may be more expensive than driven piles, due to the method of construction and installation, but with generally the same results as a driven pile. In addition, micropiles must be anchored well into sound bedrock to provide an appropriate pile capacity.

Based on the above assessment, it is Englobe's opinion that H-piles driven to refusal is an acceptable deep foundation design at this site due to the anticipated refusal depths, low bearing resistance soils, and high groundwater table encountered at this site. Driven steel pipe piles could also be considered as a variable design if submitted by a contractor.

Other proprietary methods, such as rammed aggregate piers supporting conventional shallow foundations, could also be considered. However, these designs are provided by the supplier once they review the proposed structure in combination with the borehole data. As discussed with the client, preliminary conversations with ground improvement contractors identified that additional overburden investigation(s) would be required in order to assess the viability of their potential solutions.

Englobe recommends the use of shallow conventional foundations only under the assumption that localized dewatering is undertaken in such a manner as to prevent subgrade disturbance and allow for the construction of engineered fill pads. Otherwise, deep foundations shall be used. The choice between conventional shallow foundations requiring localized dewatering in numerous small excavations, micro-piles, and driven piles would come down to a cost analysis and which option is more cost effective.

The following sections summarize the general geotechnical parameters to be used for design, foundation recommendations, excavation and backfill recommendations, frost considerations, and geotechnical concerns/discussion for construction of the new buildings.

4.3 Frost Protection

The estimated frost depth penetration for the area of the subject site is:

- ±1.8 m below exposed asphalt surfaces or for isolated, unheated foundations;
- ±1.6 m for exterior footings in a heated structure below exposed surface (i.e., adjacent sidewalks, etc.);
- ±1.4 m for naturally insulated (i.e., snow cover) exterior footings for a heated structure.

It is noted soil types that have a moderate to high susceptibility to frost heaving were encountered at this site.

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

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

Foundations can be founded at a higher elevation 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 or unheated structures founded on sandy soil with a minimum of 300 mm of final soil cover overtop the insulation.

For the heated condition where footings are supported within the anticipated depth of frost penetration, 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 foundation edge a minimum of 1.2 m.

For the unheated foundation situation, for example a raft slab at grade or shallow footings for unheated structures established within the frost zone, synthetic insulation, minimum 60 mm thick, should be placed below the foundation/slab and then extend outwards horizontally beyond the foundation edge a minimum of 2.4 m. The horizontal distance can potentially be reduced based upon the depth of footing (see Appendix F for typical heated and unheated insulation details).

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.).

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 soil subgrade must be protected from frost at all times during foundation excavation and construction operations. Should freezing temperatures occur during construction, the founding subgrade must be insulated (straw, insulated traps, etc.) against frost until such a time that footings are adequately protected (soil cover, insulation 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.

4.4 Site Development

Based on the borehole data, the natural overburden generally consists of alternating layers of silty clay/clayey silt and sand. The SPT “N” values and in-situ vane results returned from the strata indicate that the overburden is generally of low strength from a bearing capacity point of view to a depth of at least 14.9 m where DCPT operations were terminated. The presence of weak soils poses a potential bearing issue in that the native material is not strong enough as-is to support the increased loads from the proposed structure and may result in significant settlement. As such, ground improvement methods are recommended for this site.

Regardless of the method of site improvement and construction, it must be noted that the natural overburden soils are susceptible to long-term consolidation settlement if subjected to heavy area loads, such as raising the grade above original grade over a large area. As such, provided grade raises are limited to 300 mm or less, long-term consolidation settlement will not be a concern.

In order to improve the bearing capacity, it is recommended that a pad of engineered fill be placed below all footings within the area of influence. However, considering the shallow groundwater, removal of the weak native material to replace it with engineering fill may require significant dewatering operations, depending on the final grades and excavation depths. As noted, in order to avoid the need for a full site dewatering operation (vacuum well pints, sheet piling, etc.) and potentially the need for a PTTW, the contractor will need to plan for numerous small foundation excavations, each dewatered individually. To protect the available natural bearing at each foundation, the contractor will need to install dewatering operations to a suitable depth at each small foundation excavation, and initiate dewatering several days in advance of excavating to the subgrade levels. Further, the contractor will need to ensure that they maintain daily dewatering quantities below regulatory limits.

It is re-emphasized that special care must be taken to protect the subgrade and the available natural bearing capacity.

As noted, the anticipated founding subgrade will be variable across the Site.

Sand subgrades are typically straightforward to dewater as the process tends to affect the subgrade fairly quickly, and improvement of the subgrade by proofrolling allows the preparation of a suitable subgrade with a consistent in-situ density.

Dewatering in clay is somewhat more challenging as the groundwater will be slow to discharge. However, assuming an expedient excavating and subgrade preparation operation, standard dewatering techniques should be suitable considering the anticipated shallow and small excavations. Clay subgrades tend to be straightforward to prepare, provided the contractor uses a ditching/toothless bucket at the subgrade level to produce a smooth surface and protects the surface of the subgrade immediately upon exposure, and following approval, either with a granular working mat or a mud slab.

Silt subgrades however, including silty sand to sandy silt, and clayey silts are significantly more challenging to prepare. Dewatering of the silt subgrades must be undertaken well in advance of the excavation reaching the founding subgrade. Silt is dilatant in nature and when wet tends to disturb easily from vibrations as small as foot traffic. The disturbance of a silt subgrade is visually apparent by a “liverish” or rolling condition. Once disturbed, the silt subgrade loses much of its natural bearing capacity and must be corrected, either through aggressive dewatering and/or removal and replacement with either engineered fill or low strength concrete. However, further excavation into silt that is wet may simply exacerbate the disturbed condition. It is critical that silt subgrades be adequately dewatered prior to excavating, and continuously through the foundation construction process. Once exposed and approved, it is prudent to protect silt subgrades quickly with either a mud slab or an engineered fill working mat.

4.5 Foundation Recommendations

4.5.1 Shallow Foundations

The natural overburden in the area of the proposed structure consists of alternating layers of sand and silty clay with an anticipated shallow groundwater table. As noted, even with ground improvement, bearing will be limited due to depth of the generally loose sand and silt overburden. The native material will need to be removed from below the footings to a sufficient depth in order to place a pad of engineered fill. Based upon the geotechnical data collected, the proposed foundations can be supported on a pad of engineered fill overlying the natural subgrade provided the natural subgrade is properly protected and prepared.

In order to improve the bearing capacity below foundations for the proposed two storey structure, a pad of engineered fill will need to be constructed. The pads of engineered fill placed below the spread and/or strip footings shall be 600 mm thick. Excavations will be required to reach some 1.5 mbgs assuming that footings are established at 900 mm below finished grade and insulated. As noted, in order to construct the pad of engineered fill will require dewatering and in order to prevent the potential need for a PTTW, the excavations will need to be small and dewatered individually. At 1.5 m below existing grade, or approximately elevation ±99.5 m, the subgrade may be silt, clayey silt, or sand.

A test area should be completed to determine the feasibility of placing engineered fill over site materials and the effectiveness of the contractor's methods to protect the natural subgrade.

All foundation excavations and bearing surfaces should be inspected by a qualified geotechnical engineer 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. All organics and deleterious materials must be removed from the subgrade surface prior to placement of Engineered Fill at the foundation locations. If adequate dewatering has been undertaken, and the type of subgrade soil will benefit from improvement prior to pad construction (i.e., sands), the subgrade surface will need to be proof-rolled using a 5000 series Wacker (or equivalent/better) under the supervision of a geotechnical engineer to a consistent in-situ density and the maximum achievable based on field tests. Soft spots identified are to be subexcavated and replaced with granular backfill or non-shrink fill.

It is emphasized that fine-grained or loose sand subgrades can be easily disturbed during foundation construction operations, especially when wet. It is critical that the groundwater level be established in advance of excavating operations such that dewatering operations can be planned for and implemented. The working subgrade must be dewatered to a depth well below the subgrade working level in order to maintain the available natural bearing capacity, typically 500 mm to 1.0 m, and the dewatering maintained until backfilling and foundation construction is complete. If disturbed, the subgrade can lose much of its available natural bearing capacity. The contractor should also minimize worker traffic within the foundation areas, in order not to disturb the subgrade surface. To minimize disturbance of the native soils, the native soils shall be protected from ingress of water at all times during construction. In addition to dewatering, consideration should be given to using a toothless bucket at the working subgrade level. A low strength mudslab shall be placed immediately following subgrade exposure and approval.

If the founding subgrade is excessively disturbed during excavation and foundation construction operations, it may have to be subexcavated and replaced with engineered fill or non-shrink fill.

Due to the variable subgrade type to be encountered and the relatively low strength of the overburden from a bearing capacity, a biaxal or multi-axial geogrid (e.g., TriAx® Geogrids) shall be installed in the base layer of engineered fill. Geogrid designs are proprietary and can be provided by the supplier on demand provided that the project uses the suppliers' products.

Isolated footings outside the building footprint (i.e., light standards, etc.) can also be taken to bear on a pad of engineered fill supported on the natural soils with adequate frost protection as detailed in section 4.3.

It is noted that a minimum of 1 m of soil cover above the foundation depth is required to achieve the bearing capacities provided below.

Where specified or required to raise the founding subgrade elevation, engineered fill below the footings shall be placed within the area of influence and should consist of an imported material meeting OPSS for Granular A or Granular B Type II, compacted to 100% SPMDD. It should be noted that Granular B Type II is a manufactured material consisting of 100% crushed quarry stone. Granular B Type II can only be used where the depth of fill is greater than 3 times the maximum size of aggregate. 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.

The area of influence below the individual foundation units, in cross section, is described as a trapezoid that extends outwards, horizontally from the edges of the foundation, a minimum of 300 mm and then downwards on a 45° (1V:1H) outward angle to undisturbed native competent soil.

The design is based upon the assumption that the footings will be properly formed (i.e., earth forms are not acceptable) and any required rebar is placed in accordance with standard practices. Backfill around and above the foundations should consist of a well graded free draining Granular B Type III compacted to at least 98% of the SPMDD.

For footings on Engineered Fill with an appropriate geogrid over a mudslab on approved natural soils, Englobe suggests that the following recommendations for the ultimate limit state (ULS) and serviceability limit state (SLS) be implemented into the engineered fill pad design:

Table 4-1: Geotechnical Resistances and Reactions for Footings on Engineered Fill supported directly on Natural Subgrade.

Depth to Underside of footing (m)	Bearing Material	Maximum Footing Size (m)	Resistance at Factored ULS (kPa)	Reaction at Factored SLS (kPa)[(1)]
1.0	0.6 m Engineered Fill over Native Silty Clay / Sands	Spread Footings up to 1.5 m x 1.5 m	150	100
		Strip footings up to 1 m wide	130	50

Note:

(1) In consideration of 25 mm total and 19 mm differential settlement.

SLS values will ensure that settlement will not exceed a maximum of 25 mm. The resistance at ULS was calculated by applying load resistance factor of 0.5 according to the Canadian Foundation Engineering Manual.

4.5.2 Deep Foundations

If the procedures necessary to construct shallow foundations are not considered feasible, a deep foundation system will need to be implemented.

4.5.2.1 Driven H-Piles

It is recommended that end-bearing driven piles be considered for the support of the 2-storey structures. The bottom of the pile cap should be installed below frost depth (as indicated in Section 4.1). For end-bearing driven H-pile design, we recommend that piles be considered using the following capacities:

Table 4-2: Pile Capacities

Pile Type	Depth Of Pile Tip Below Grade (m)	Estimated Pile Tip Elevation (m)	Factored Axial Geotechnical Resistance At ULS (kN) (Per Pile)
HP310x110(1)	±60 m below ground surface (2)	±41 to ±42(3)(4)	750(5)

Notes:

[1] HP310x110 piles or better considered in geotechnical design with yield stress if 350 MPa.

[2] Depth is estimated from historical data within the area as Englobe's investigation did not encounter bedrock

[3] Driven pile should be advanced to practical refusal on hard overlying material. Since practical refusal is essentially an unyielding subgrade, a geotechnical reaction at SLS does not apply

[4] Elevation relative to a temporary local benchmark

[5] A geotechnical resistance factor of 0.4 as per Table 6.2 of CSA S6:19 CHBDC S6:19 was used to calculate the factored foundation resistances.

Driven piles may be designed using the capacity presented in the table above if they are taken to practical pile refusal (on potential bedrock or hard overlying material). Practical refusal is normally defined as a final set of not less than 15 blows per 25 mm.

If a pile group is used, piles should be spaced at least three times their diameter, measured centre-to-centre. The interaction of pile groups must be considered from a settlement and lateral pile capacity perspectives. If pile groups are considered for this project, Englobe should be contacted to provide additional design parameters.

The structural design of steel piles must conform to the requirements of the National Building Code of Canada, and to Section 19 of the Canadian Foundation Engineering Manual. Driving of steel H-piles is expected to be generally straight forward at this site. Driven steel piles may encounter hard driving and/or refusal soon after encountering the dense refusal layer. If driven unprotected under these conditions, the pile toe may deform to an unacceptable extent. For better control during pile driving and to reduce the risk of pile damage during driving, the toes of the pile should be protected by reinforcing the pile tip with rock point such as Oslo point, Titus H bearing point, or APF Hard Bite that can assist in seating the piles within the refusal material. Piles shall be installed in accordance with OPSS.MUNI 903.

The pile installation operations should be monitored on a full-time basis by qualified geotechnical personnel to ensure uniformity of set, record pile toe and cut off elevations, and to check pile condition, alignment, splices, and plumbness.

In addition, the 'set' criteria for pile driving can be also determined utilizing pile dynamic analyzer (PDA) testing for pile inspection. Performing PDA analyses during driving of the piles not only assists with establishing appropriate driving criteria to seat the piles in suitable refusal material but also in assessing potential damage to piles during driving. Further, with PDA analysis shallower refusal conditions may be encountered and thus save cost in materials and time. Using the Hiley Dynamic Pile Driving Formula (MTO Standard SS103-11) is not recommended as Hiley Formula may provide predictions with relatively large amount of variation for piles driven to refusal on bedrock. In addition, the PDA testing should be completed on a minimum of 10% of the piles.

It is recommended that the axial capacity and the driveability of a driven steel pile be confirmed using dynamic analysis (e.g., Wave Equation Analysis) based on driving response. Such analysis is also useful for hammer size selection and establishing termination criteria for pile driving. As a guide and in the absence of local experience, the rated energy of the hammer should be limited to a value of 6×10^6 J (Newton-metre) times the cross-sectional area of the pile. For recommendations for the driving cap and capblock, see Section 19.3.5.3 of the Canadian Foundation Engineering Manual, 4th Ed. (2006). Provisions should be made to retap the initial piles to confirm the set after adjacent piles have been driven. If it is found the set of the piles is not altered by retapping, then this procedure may be discontinued.

If pile testing (static or dynamic (PDA) load testing) is conducted, a higher geotechnical resistance factor can be applied to achieve a higher factored shaft and base resistance. As such, pile testing can help to optimize the pile design and to reduce foundation costs.

4.6 Slab on Grade

Slab on grade construction can be used at this site. As noted, it is understood that grade raises are not expected and finished floor elevation will be near or less than 300 mm above existing grade. Provided grade raises are limited to 300 mm or less, settlement is expected to be within normally tolerable limits for structures supported on shallow foundations on soil.

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 (after replacement of the existing 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. Once approved, the subgrade must be proof-rolled to a consistent in-situ density as proven through on-site testing. 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, compacted to a minimum 98% Standard Proctor Dry Density (SPDD) or better. OPSS.MUNI 1010 Granular A or Granular B Type II can also be considered. 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.

4.7 Earthquake Parameters

The design peak ground acceleration (PGA) and Peak Ground Velocity (PGV) for the site were calculated as 0.0805 g and 0.0955 m/s, respectively, with a 2% probability of exceedance in 50 years based on the interpolation of the 2020 National Building Code Canada (NBCC) Seismic Hazard calculation, attached in Appendix E.

Considering the geotechnical values, and based on 2020 NBCC, Table 4.1.8.4A, Site Classification for Seismic Site Response, the subject site would have Site Class E (Site Designation X_E).

4.8 Drainage

It is understood that the building will not have a basement or other below grade structures.

For those structures that do not have a basement, underslab drainage should not be necessary, provided the top of floor slabs are a minimum of 300 mm above exterior grade.

Considering the shallow groundwater encountered at site, it is recommended that full perimeter footing drains (e.g., weeping tile) be installed at the toe of the foundation (see Appendix F). The drainage system must be properly sloped towards a suitable outlet to provide continuous positive drainage. It is strongly recommended that a geotextile separator (i.e., Terrafix 270R or equivalent) be placed between the natural subgrade and the backfill material, in addition to the perforated pipe being wrapped in a filter cloth to prevent the buildup of fines within the drainage system.

The surface of the finished grade around the exterior of the building should be relatively impermeable and contouring of the perimeter exterior grade surface must direct all surface waters away from the structure.

4.9 Excavation, Dewatering, and Backfilling

Based on the Occupational Health and Safety Act Regulations for Construction Projects, the soil at this site is classified as:

a) Type 3 (native silty clay)

b) Type 4 (native sand)

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 at least 2H:1V or possibly shallower depending upon the method of dewatering employed, or possibly sheeted, and maintained under continuous observation.

A dry subgrade condition must be maintained at all times during foundation construction until both footing and foundation wall construction, and backfilling, are a sufficient height above the prevailing water table (i.e., at a minimum 1 m). At the time of the fieldwork, the groundwater table was encountered as shallow as 0.6 m below existing grade. The Contractor must undertake to establish the groundwater level in advance of the construction operations such that adequate groundwater control plans can be developed.

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). An EASR or PTTW is required by the Ministry of Environment, Conservation and Parks if the daily taking of groundwater exceeds 50,000 L or 400,000 L per day, respectively.

Standard groundwater control techniques through the use of sump holes with pumps can be used at this site although the area of influence will be limited, likely requiring multiple sumps in the area of excavation. As noted, to prevent he need for a PTTW, the contractor shall plan for small individual foundation excavations, dewatered separately and monitored to ensure quantities do not exceed the regulatory requirements.

Temporary construction groundwater control is typically undertaken using oversized excavations (relative to the foundation unit) and installing perimeter/interior drains/ditches leading to a sufficient number of strategically placed filtered sump holes located in the base of the excavation outside the area of influence of engineered fill and/or foundations. It is noted that the efficiency of conventional sump holes to control the groundwater depends highly upon the number of sumps, the depth of their base below the ultimate subgrade level, method of construction (i.e., cased and filtered sump hole versus a pump at the base of the excavation), and their spacing. In our experience, to be efficient at groundwater control, conventional sump holes should not be placed more than 5 m apart, preferably less, although placement is highly dependent upon soil types (permeability, etc.) and conditions, depth of sump holes, extent/depth of drains/ditches leading to the sumps, as well as the intent of the project. Where greater draw down is required, a more sophisticated dewatering system will be required that will have to be developed by a qualified dewatering subcontractor. In order to be effective any dewatering operation must be started well in advance of the excavating operations and be run continuously throughout the subsurface construction operations.

The Contractor must undertake to control surface water that develops from precipitation or snow melt that may become perched in the excavations during excavating operations. The groundwater control program designed by the Contractor should account for this during construction operations.

It must be emphasized that, when wet, loose or silty soils (such as encountered at this site) can be easily disturbed through excavation operations, foot traffic, etc. and such disturbed soils can lose a significant amount of the natural bearing. To minimize the potential for disturbance, the groundwater must be drawn down a sufficient depth below the base of the excavation (i.e., 500 mm to 1 m).

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 (i.e., sump holes and pumps, drainage ditches, vacuum well points), to construction of system (sump hole details, placement, etc.), to operation of system, etc.

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. It is strongly recommended that a ditching bucket or a bucket with a blade across the teeth be used in order to prepare a smooth subgrade surface.

A geotextile separator (i.e., Terrafix 270R or equivalent) shall be placed between the granular subbase and any areas of finer grained soil (i.e., silt or clay).

The native material was not found to consistently meet any OPSS Form 1010 specification and can therefore only be used in areas of landscaping or elsewhere where movement of the ground surface is not of concern, unless a stockpiling and quality control segregation program is undertaken to identify those sands that may meet for Select Subgrade material.

Any soil to be removed from the Site will 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.).

4.10 Pavement Design

4.10.1 Flexible Pavement Design

The production of a long-lasting, quality pavement structure is highly dependent upon several factors. The pavement structure that supports and distributes the traffic load consists of three separate layers: the subbase, base, and wearing surface. This, in turn, is supported by the subgrade. The long-term performance of the pavement structure is highly dependent upon the uniformity of these layers. A uniform subgrade cross-section must be maintained below the entire pavement structure, free of depressions, and properly sloped towards adequate drainage facilities.

Generally, the overburden at this site in the area of the proposed access route and parking lot consists of sand to clayey silt deposits within the zone and anticipated depth of frost penetration. *Based on the gradation analysis, these deposits have a low to high susceptibility to frost heaving.*

The overburden present in the depth of frost penetration at this site is considered to have a high susceptibility to frost heaving. The subgrade below the pavement structure must have a **uniform** (i.e., consistent subgrade type, moisture and density conditions, etc.) cross section within the depth of frost penetration to reduce differential heaving. As noted, the soil conditions are variable between the two boreholes advanced in the parking lot area. Proper subgrade tapers must be provided between areas where the native subgrade is encountered at different depths below existing grade, where different types of subgrade are encountered, or where service trenches are installed below the pavement structure, to provide a uniform subgrade transition to minimize the risk of differential heaving. Proper frost tapers impart a gradual heave and as such are less noticeable. Standard frost tapers for highway construction are established at a minimum of 10H:1V slope for centerline culverts etc. Considering the low anticipated speeds in the parking lot these tapers may be reduced to 5H:1V, however, it must be noted that differential heaving will be more noticeable where the steeper taper is used. If service trenches are installed outside of the asphalt area, frost tapers will not be required.

Emphasis is again provided as to the sensitive nature of this type of soil. Every effort must be made to prevent disturbing the founding subgrade during excavating and construction operations.

All deleterious surficial materials (i.e. fill, organic soil, disturbed soil, etc.) should be stripped from below the area of influence of the pavement structure to allow construction of the pavement structure described below. Some isolated areas may require deeper excavations if areas of weak/poor subgrade become evident during construction.

Once stripped, the surface of the exposed subgrade should be crowned at a minimum of 3% cross fall gradient towards a positive drainage system (i.e. subdrains, ditch etc.). Once the site is stripped of deleterious materials down to the subgrade level, the subgrade must be approved by a qualified member of this firm. Once the subgrade is approved, engineered fill may be required to raise the site from approved subgrade to underside of the pavement structure. If backfill is required it is recommended that, at a minimum, an imported material meeting OPSS for a well graded SSM be used.

If service trenches must cross below paved areas (for electrical wiring, servicing, sewer, etc.) and the trench invert is below the subgrade level, but within the anticipated depth of frost penetration, frost tapers shall be applied to the trench sidewalls or the area of the trench insulated with synthetic insulation. If the utility trench is below the depth of anticipated frost penetration, the utility should be properly bedded in granular as per manufacturers specifications, however, the trench should be backfilled, up to subgrade level, using the native trench materials (provided they are at an appropriate moisture content and can be adequately compacted) within the depth of frost penetration to minimize the risk of differential heaving. Service trenches established in fine grained subgrades with low permeability must have drainage established at the lowest possible elevation that will still allow gravity drainage. This is to prevent the buildup of water within the service trench, especially within the depth of frost penetration.

Provided the subgrade is properly prepared and is uniform, we recommend the following pavement structure.

Table 4-3: Recommended Pavement Structure Components

Pavement Structure Component	Access Routes & Heavy Duty (I.E. Bus Routes, Access Roads, Delivery, Etc.)	Parking Area Light Duty (I.E. Cars Etc.)
Surface Course Asphaltic Concrete HL-3 (OPSS 1150)	50 mm	50
Base Course Asphaltic Concrete HL-8 (OPSS 1150)	40 mm	-
Base (OPSS.MUNI 1010 Granular A)	150 mm	150 mm
Subbase		

Pavement Structure Component	Access Routes & Heavy Duty (I.E. Bus Routes, Access Roads, Delivery, Etc.)	Parking Area Light Duty (I.E. Cars Etc.)
(OPSS.MUNI 1010 Granular B Type I or III) or (OPSS.MUNI 1010 Granular B Type II)	600 mm or 450 mm	600 mm or 450 mm

The binder course has been specified to be thinner than the surface course to allow the surface course to be of uniform thickness when paving beyond those areas requiring a binder course. If heavy duty design is used throughout the parking lot and access routes, the thickness of the binder course and surface course noted in the table should be reversed.

The importance of draining the pavement structure (granular materials) below the parking/access areas cannot be stressed enough. Ditches or other drainage facilities should be provided to the paved areas. To provide positive drainage of the granular base and subbase material below the paved areas, the invert of the ditches should be placed at a minimum of 0.5 m below the underside of the subbase (i.e., Granular B Type I or II) level.

Ditches require regular maintenance and cleaning to ensure positive drainage. If perforated drainage pipe is used for subsurface drainage, the invert of the drainage pipe should be located at the underside of the subbase layer.

Based on the variable fines content in the subgrade material encountered in the borings, the perforated drainage pipe should be surrounded by an approved porous geotextile membrane or equivalent granular filter. The ditches and/or perforated drainage pipe must have a positive gradient towards an outlet or catch basin that will provide continuous drainage.

4.10.2 Compaction

All OPSS.MUNI 1010 Granular A (base course) and OPSS.MUNI 1010 Granular B Type I/III or II (subbase) material should be compacted to a minimum of 100% Standard Proctor Maximum Dry Density (SPMDD). Imported fill (SSM) should be compacted to a similar density where possible above the prevailing ground water table.

4.10.3 Service Life

A functional design life of 15 years has been used to establish the pavement recommendations. This represents the number of years to the first rehabilitation, assuming and provided that routine and regular maintenance is carried out.

The long-term performance of the pavement structure is highly dependent upon the subgrade support conditions. Stringent construction control procedures should be maintained to ensure that uniform subgrade moisture and density conditions are achieved. In building, the need for adequate drainage cannot be overemphasized. The finished pavement surface and underlying subgrade should be free of depressions and should be sloped to provide effective drainage. Surface water should not be allowed to pond adjacent to the outside edges of pavement areas.

4.11 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:

for deep and shallow foundations: design review and full-time monitoring during construction; and 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 the borehole location, 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 foundations:

Confirm that materials and methods meet specifications;

Inspect foundation subgrades for bearing review;

Inspect excavations;

Review foundation installation/testing methods;

Review compaction testing records; and

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.;

Obtain additional samples to confirm materials and SPMDD for compaction requirements;

Inspect subgrade prior to any fill placement;

Monitor excavation side slopes;

Quality control/assurance of granular and select fill material; and

Review compaction testing records.

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 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
Drawing No. 3	Conceptual Site Plan

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No.	Version	Date	By	Verif	Appr.
01	Final	2024/03/27	DMc	RT	JRB

Ontario Aboriginal Housing Services

Geotechnical Investigation Proposed Housing Development

243 River Road, Sault Ste Marie, Ontario

Key Plan (Macro)

ENGLOBE

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

Discipline: Geotechnical
Scale: Not To Scale
Date: 2024/03/27
Page setup: Macro
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Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
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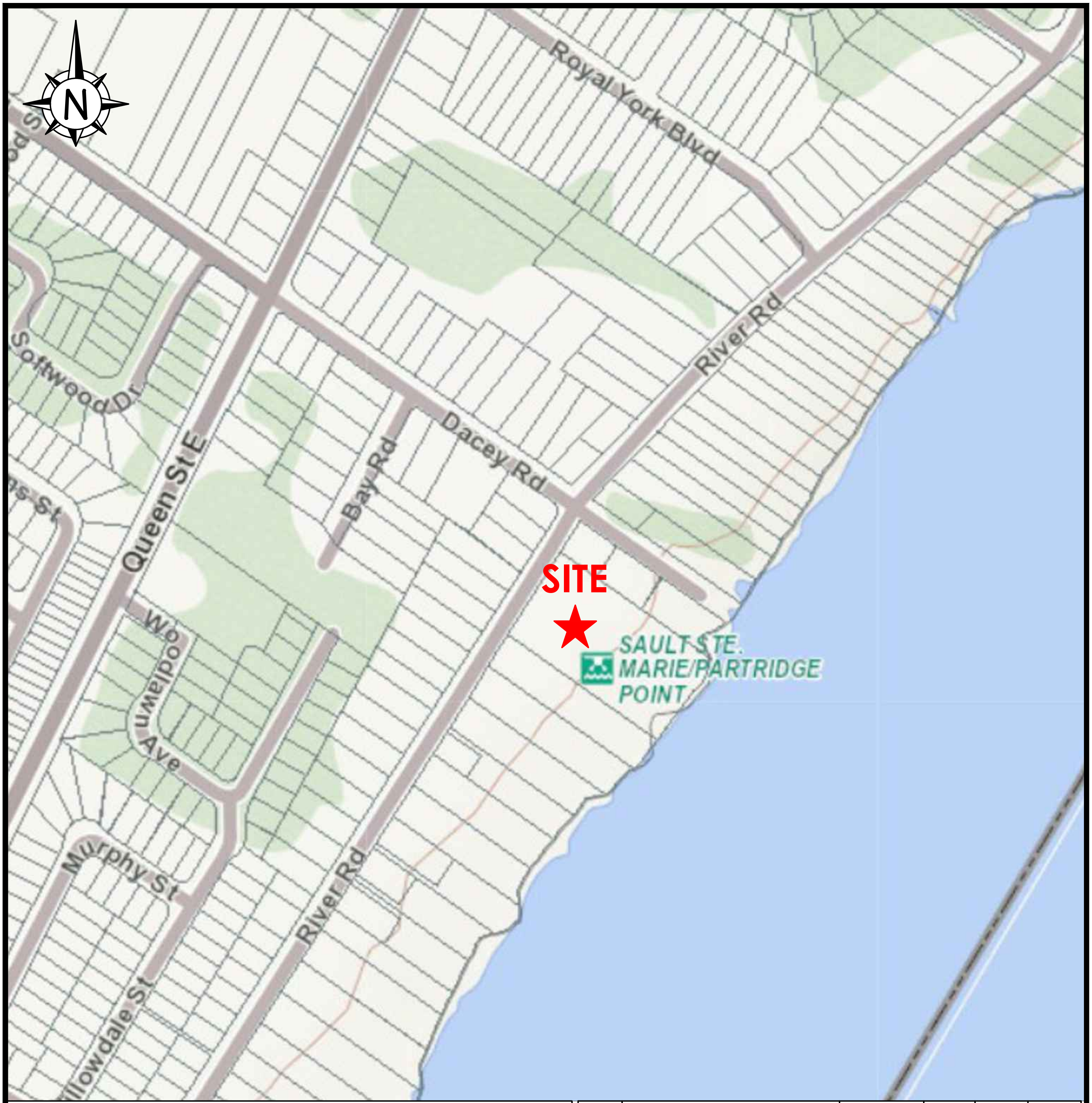

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01	Final	2024/03/27	DMc	RT	JRB
No.	Version	Date	By	Verif	Appr.

Ontario Aboriginal Housing Services

Geotechnical Investigation
Proposed Housing Development

243 River Road, Sault Ste Marie, Ontario

Key Plan (Micro)

ENGLOBE

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

Discipline: Geotechnical
Scale: Not To Scale
Date: 2024/03/27
Page setup: Micro
Paper size: 8.50 X 11.00 in.
Prepare by: DMc
Verify by: RT
Drawn by: DMc
Approval by: JRB
Drawing no: 1b
Register no.: ---------

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
EL	02312415	---	GE	-	--	--	----	01

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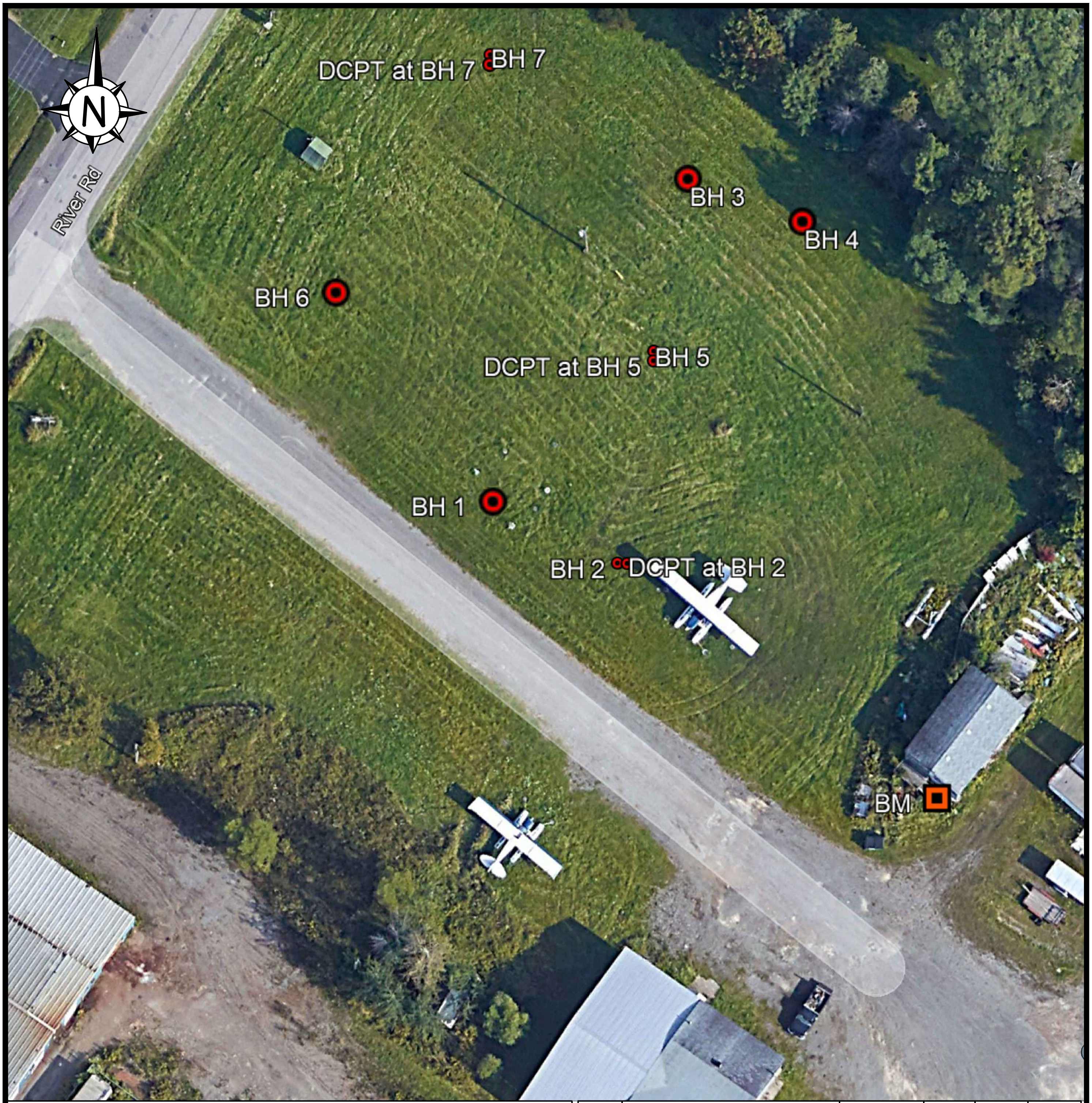

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No.	Version	Date	By	Verif	Appr.
01	Final	2024/03/27	DMc	RT	JRB

Ontario Aboriginal Housing Services

Geotechnical Investigation
Proposed Housing Development
243 River Road, Sault Ste Marie, Ontario

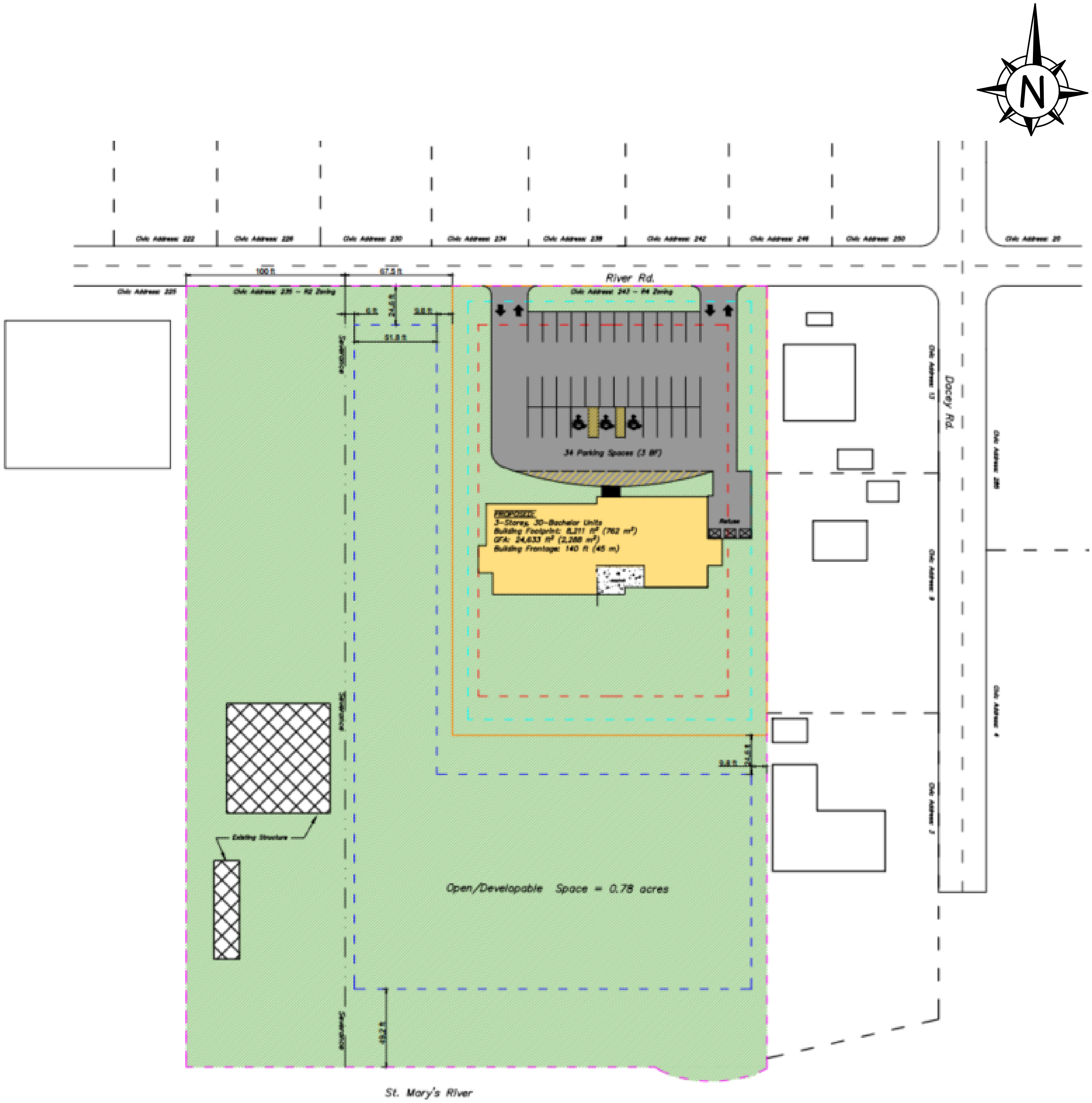
Borehole & DCPT Location Plan

ENGLOBE

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

Discipline: Geotechnical
Scale: Not To Scale
Date: 2024/03/27
Page setup: BH Plan
Paper size: 8.50 X 11.00 in.
Prepare by: DMc
Verify by: RT
Drawn by: DMc
Approval by: JRB
Drawing no: 2
Register no.: ---------

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
EL	02312415	---	GE	-	--	--	----	01



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01	Final	2024/03/27	DMc	RT	JRB
No.	Version	Date	By	Verif	Appr.

Ontario Aboriginal Housing Services

Geotechnical Investigation
Proposed Housing Development
243 River Road, Sault Ste Marie, Ontario

Conceptual Site Plan

ENGLOBE

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

Discipline:	Geotechnical	Prepare by: DMc	Verify by: RT
Scale:	Not To Scale	Drawn by: DMc	Approval by: JRB
Date:	2024/03/27	Drawing no: 3	
Page setup: Site Plan	Paper size: 8.50 X 11.00 in.	Register no.: ---------	

Man.	Project	Otp	Project	Phase			Electronic ref.	Rev.
EL	02312415	---	GE	-	--	--	----	01

Appendix B
Borehole Logs

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure No. 2 to 6	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)

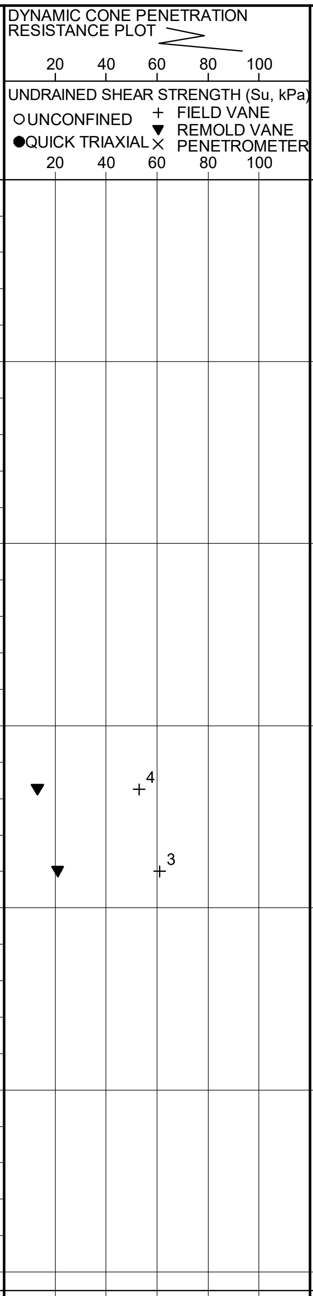
METRIC

RECORD OF BOREHOLE NO. 1

ENGLOBE

REFERENCE	02312415.000	DATUM	TBM	LOCATION	See Borehole Location Plan; Appendix A, Dwg. No. 2
ORIGINATED BY	ES				
PROJECT	GI - 243 River Road, Sault Ste. Marie	BOREHOLE TYPE	Geoprobe 7822DT Drill Rig - Hollow Stem Auger	COMPILED BY	DMc
CLIENT	Ontario Aboriginal Housing Services	DATE (Started)	January 31, 2024	TIME	
		DATE (Completed)	January 31, 2024	(Completed)	9:35:00 AM
CHECKED BY	RT				

ELEV DEPTH	SOIL PROFILE – DESCRIPTION (see Enclosure No. 1)	SAMPLES NUMBER	TYPE	"N" VALUES
101.0				
0.0	TOPSOIL - 250 mm	1A	SS	5
100.8 0.3	SILT - brown, loose	1B		
		2A	SS	3
99.9 1.1	CLAYEY SILT - brown, very soft	2B		
99.4 1.6	SAND - trace silt, grey, very loose	3A	SS	1
		3B		
98.9 2.1	SILTY CLAY - brown, very soft	4A	SS	WH
		4B	ST	
	- sand seam	5	SS	WH
96.4 4.6	SAND - trace silt, brown, very loose	6	SS	WH
	- silty clay seam	7	SS	WH
94.9 6.1	End of Borehole at 6.1 m bgs			



GROUND WATER CONDITIONS

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)

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)	1/31/24 10:05:00 AM	0.7	1.3
2)	1/31/24 12:35:00 PM	0.65	1
3)		-	-

MEL-GEO 2312415 - RIVER ROAD.GPJ MEL-GEO.GDT 3/8/24

ENCLOSURE NO.:3 (Pg. 1 of 2)

METRIC

RECORD OF BOREHOLE NO. 2

ENGLOBE

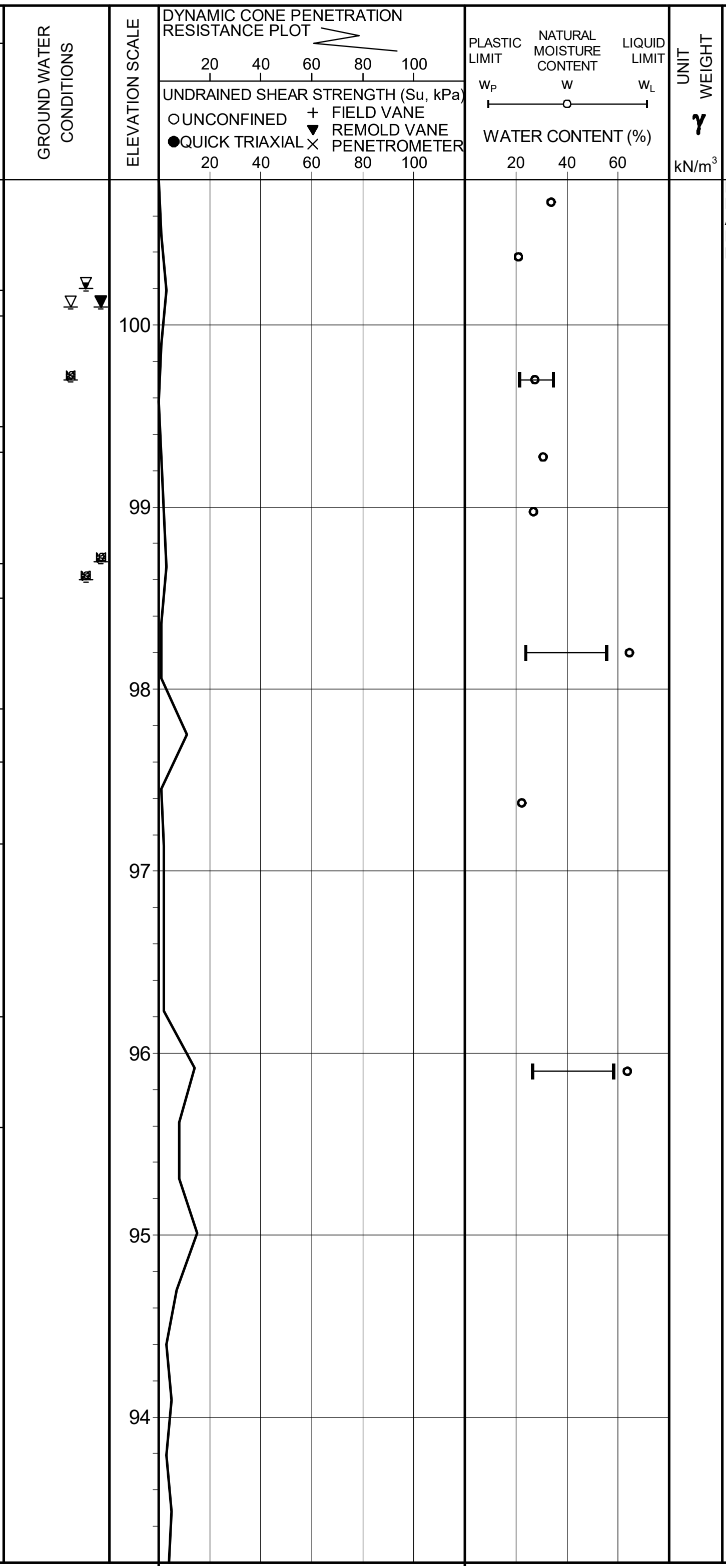
REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services DATE (Started) January 30, 2024 DATE (Completed) January 31, 2024 TIME (Completed) 4:30:00 PM CHECKED BY RT

ELEV DEPTH	SOIL PROFILE DESCRIPTION (see Enclosure No. 1)	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
100.8						
0.0	TOPSOIL - 250 mm		1A	SS	6	Dynamic Cone Penetration Test Advanced 1.5 m East of Original Borehole
100.6 0.3	SAND - trace silt, brown, loose		1B			
100.1 0.8	CLAYEY SILT - brown, very soft		2	SS	WH	0 5 68 27
99.3 1.6	SAND - some silt, brown, loose		3A 3B	SS	5	0 87 (13)
98.5 2.3	SILTY CLAY - very soft		4	SS	WH	
97.6 3.2	SAND - trace silt, brown, wet, loose		5	SS	8	0 98 (2)
96.5 4.3	SILTY CLAY - brown, very soft		6	SS	WH	
95.6 5.2	SAND - trace silt, brown, very loose					
94.7 6.1	End of Sampling at 6.1 m bgs					

Continued Next Page



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) 1/30/24 5:25:00 PM	0.7 ▽	1.1
2) 1/31/24 8:25:00 AM	0.6 ▽	2.2
3) 1/31/24 12:50:00 PM	0.7 ▼	2.1

ENCLOSURE NO.:3 (Pg. 2 of 2)

METRIC

RECORD OF BOREHOLE NO. 2

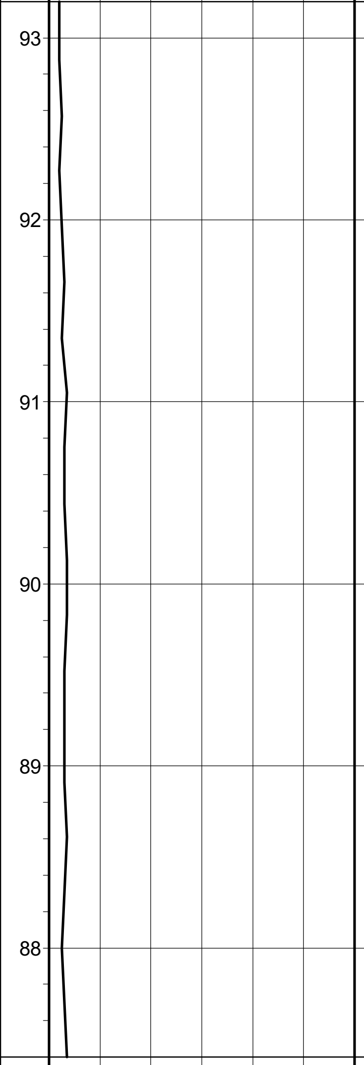
ENGLOBE

REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services DATE (Started) January 30, 2024 DATE (Completed) January 31, 2024 TIME (Completed) 4:30:00 PM CHECKED BY RT

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1) Continued from Previous Page	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 UNDRAINED SHEAR STRENGTH (Su, kPa) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL ▼ REMOLD VANE × PENETROMETER 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)
87.4 13.4	End of Borehole at 13.4 m bgs										



ENCLOSURE NO.:5 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 3

REFERENCE 02312415.000 DATUM TBM

LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2

ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie

BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger

COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services

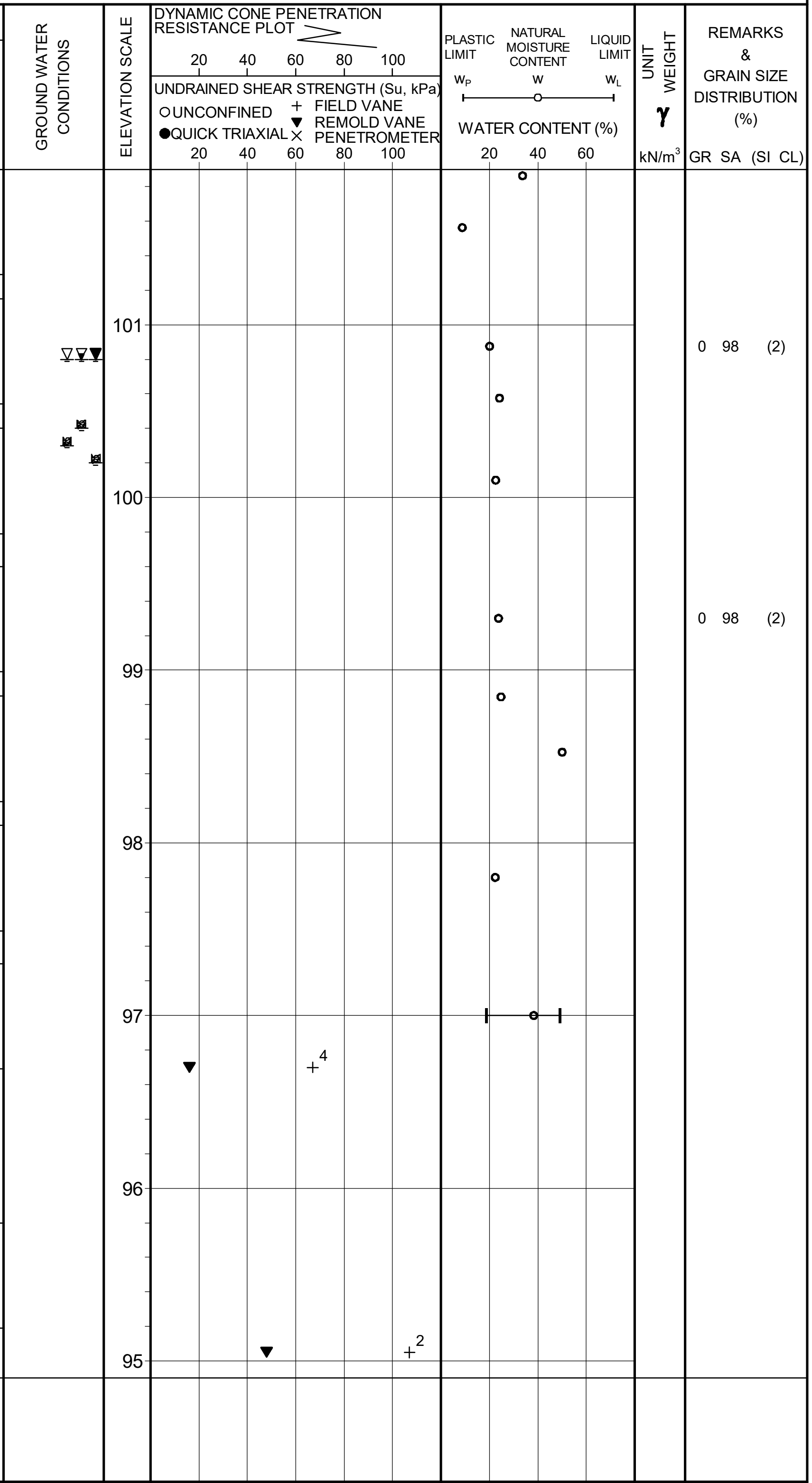
DATE (Started) January 30, 2024

DATE (Completed) January 30, 2024

TIME (Completed) 5:25:00 PM

CHECKED BY RT

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV / DEPTH	DESCRIPTION (see Enclosure No. 1)	NUMBER	TYPE	"N" VALUES	GR SA (SI CL)
101.9					
101.8 / 0.1	TOPSOIL - 75 mm SAND - trace silt, brown, loose	1A			
		1B	SS	6	
		2A	SS	2	0 98 (2)
		2B			
	- clay seam				
	- wet				
		3	SS	2	
		4	SS	2	0 98 (2)
98.8 / 3.1	SILTY CLAY - brown, very soft	5A			
		5B	SS	1	
98.3 / 3.7	SAND - trace silt, brown, wet, very loose	6	SS	2	
97.5 / 4.4	SILTY CLAY - brown, very soft to stiff	7	SS	WH	
		8	SS	WH	
94.9 / 7.0	End of Borehole at 7.0 m bgs				



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) 1/30/24 5:35:00 PM	1.1 ▽	1.6
2) 1/31/24 8:37:00 AM	1.1 ▽	1.5
3) 1/31/24 12:43:00 PM	1.1 ▼	1.7

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

METRIC

RECORD OF BOREHOLE NO. 4

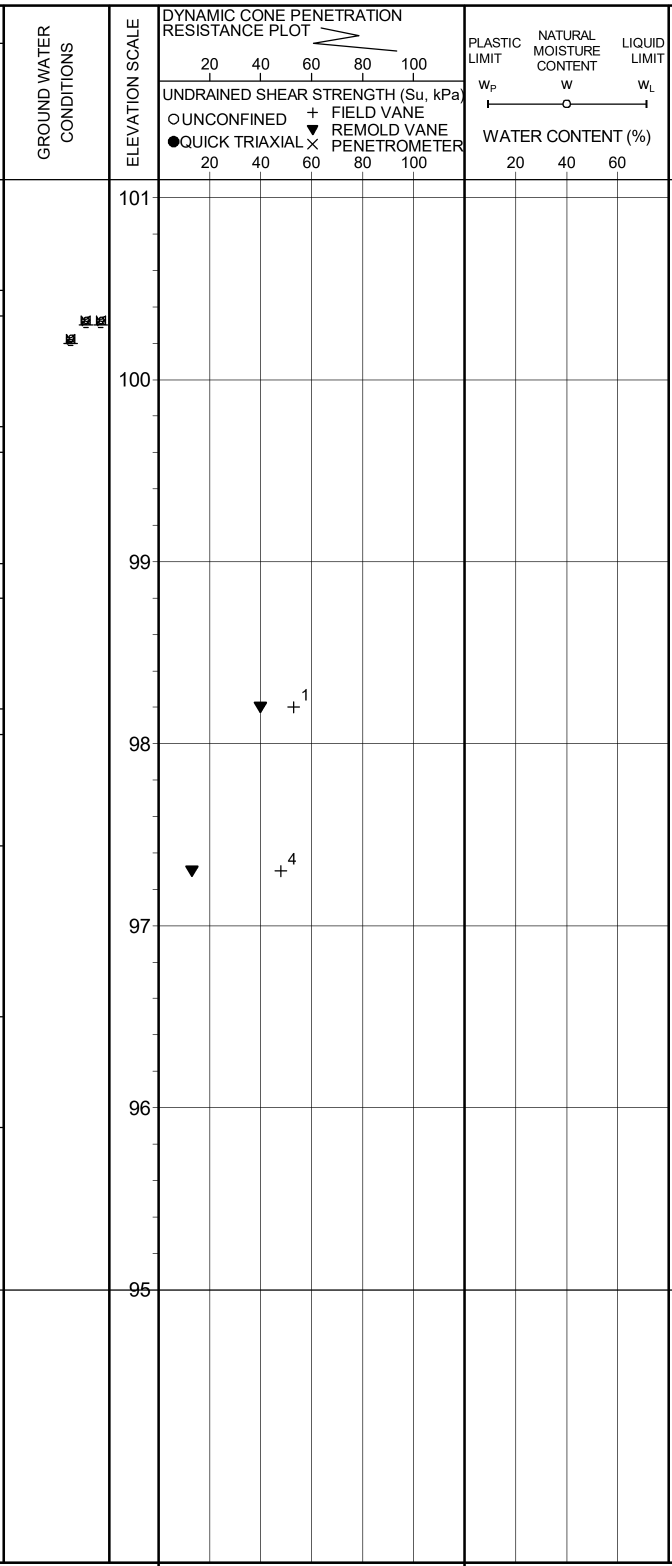
ENGLOBE

REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services DATE (Started) January 30, 2024 DATE (Completed) January 30, 2024 TIME (Completed) 3:25:00 PM CHECKED BY RT

ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
101.1 0.0	TOPSOIL - 150 mm	1A				
101.0 0.2	SAND - trace gravel, silt brown, loose	1B	SS	9		
100.4 0.8	CLAYEY SILT - brown, stiff	2A				
100.2 1.0	SAND & GRAVEL - trace silt, brown, wet, loose to compact	2B	SS	9		
		3	SS	16		
98.8 2.3	SILTY CLAY - brown, firm to stiff	4	SS	WH		
		5	SS	WH		
96.5 4.7	SAND - trace silt, brown, very loose	6A				
		6B	SS	WH		
95.0 6.1	End of Borehole at 6.1 m bgs					



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) 1/30/24 3:30:00 PM	- ▽	0.9
2) 1/31/24 8:32:00 AM	- ▽	0.8
3) 1/31/24 12:42:00 PM	- ▼	0.8

MEL-GEO 2312415 - RIVER ROAD.GPJ MEL-GEO.GDT 3/8/24

ENCLOSURE NO.:7 (Pg. 1 of 2)

METRIC

RECORD OF BOREHOLE NO. 5

ENGLOBE

REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services DATE (Started) January 30, 2024 DATE (Completed) January 30, 2024 TIME (Completed) 11:10:00 AM CHECKED BY RT

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	NUMBER	TYPE	"N" VALUES	GR SA (SI CL)
101.0					
100.9 0.1	TOPSOIL - 100 mm SAND - some silt, brown, wet, loose	1A 1B	SS	4	Dynamic Cone Penetration Test Advanced 0.9 m South of Original Borehole
100.2 0.8	CLAYEY SILT - brown, soft	2A 2B	SS	2	
99.5 1.5	SAND - some silt, brown, wet, very loose	3	SS	1	0 89 (11)
98.7 2.3	SILTY CLAY - brown, soft - sand seam	4A 4B	SS	2	
	- sand seam	5A 5B	SS	2	
96.4 4.6	SAND - trace silt, clay (layers), brown, loose	6	SS	3	0 97 (3)
	- clay seam, wet	7A 7B	SS	2	
93.4	Continued Next Page				

GROUND WATER CONDITIONS

ELEVATION SCALE

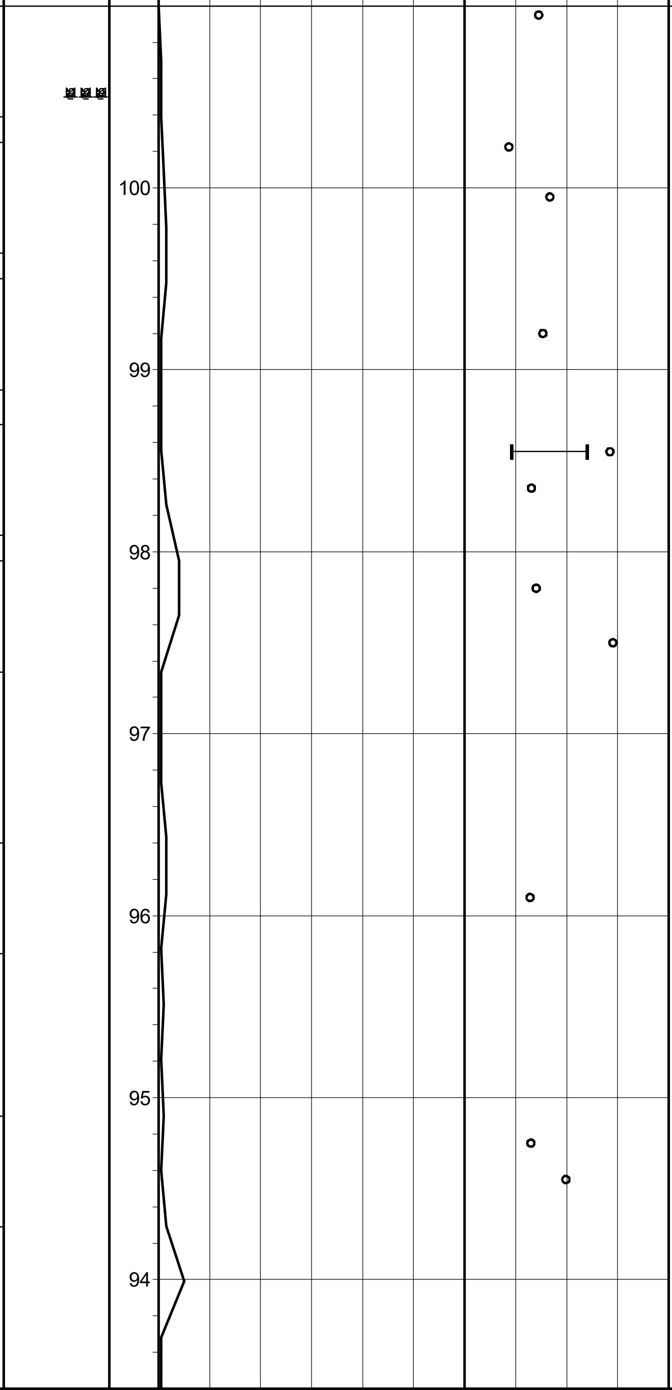
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

UNDRAINED SHEAR STRENGTH (Su, kPa)
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL ▼ REMOLD VANE
× PENETROMETER
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



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) 1/30/24 5:27:00 PM	-	0.5
2) 1/31/24 8:27:00 AM	-	0.5
3) 1/31/24 12:39:00 PM	-	0.5

MEL-GEO 2312415 - RIVER ROAD.GPJ MEL-GEO.GDT 3/8/24

ENCLOSURE NO.:7 (Pg. 2 of 2)

METRIC

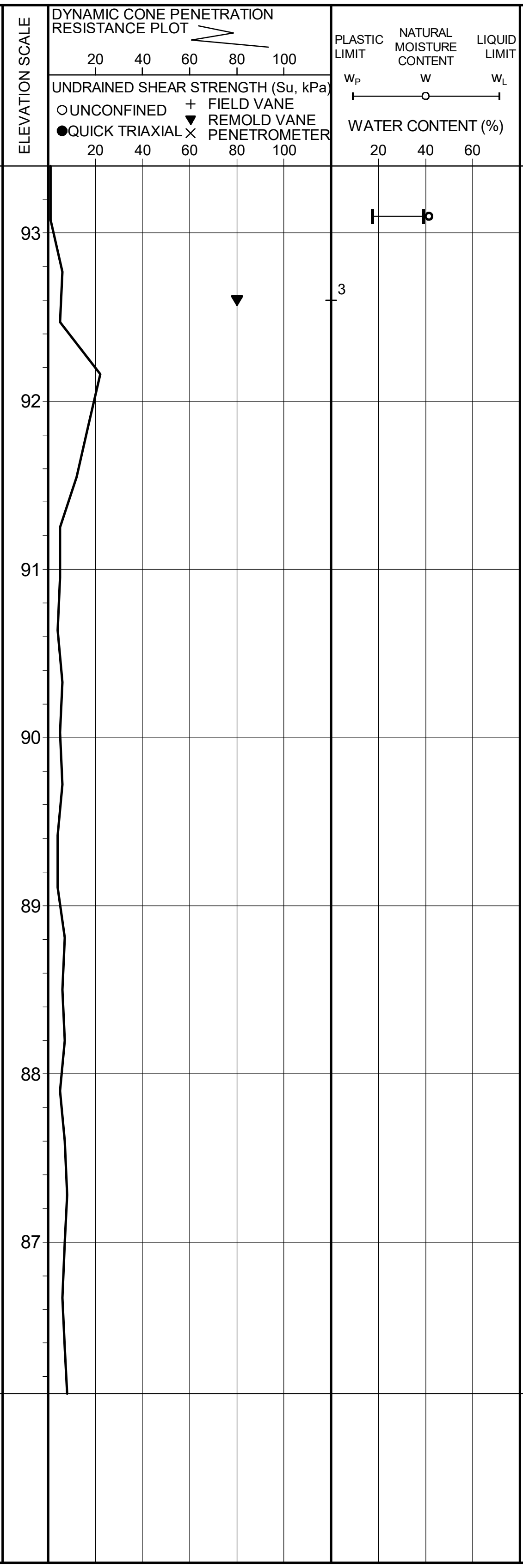
RECORD OF BOREHOLE NO. 5

REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services DATE (Started) January 30, 2024 DATE (Completed) January 30, 2024 TIME (Completed) 11:10:00 AM CHECKED BY RT

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1) Continued from Previous Page	STRATA PLOT	NUMBER	TYPE	"N" VALUES			
7.6	SILTY CLAY - brown, very soft to hard	▨	8	SS	WH			
91.9 9.1	End of Sampling at 9.1 m bgs							
86.1 14.9	End of Borehole at 14.9 m bgs							



MEL-GEO 2312415 - RIVER ROAD.GPJ MEL-GEO.GDT 3/8/24

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

METRIC

RECORD OF BOREHOLE NO. 6

REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES

PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc

CLIENT Ontario Aboriginal Housing Services DATE (Started) January 31, 2024 DATE (Completed) January 31, 2024 TIME (Completed) 12:00:00 PM CHECKED BY RT

ELEV / DEPTH	DESCRIPTION (see Enclosure No. 1)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
101.3 / 0.0						
101.2 / 0.2	TOPSOIL - 150 mm	1A	SS	6		
	SAND - trace silt, brown, loose	1B				
100.6 / 0.8	CLAYEY SILT - trace sand, brown, very soft	2A	SS	WH	▽	0 6 67 27
		2B				
99.8 / 1.5	SAND - trace silt, brown, wet, loose	3	SS	10		
		4	SS	9		
98.4 / 2.9	End of Borehole at 2.9 m bgs					

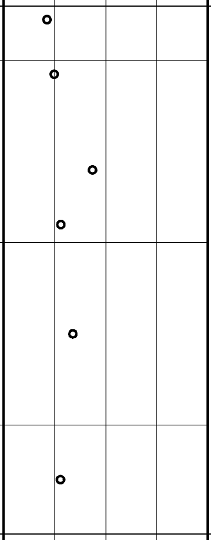
ELEVATION SCALE: 101, 100, 99

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

UNDRAINED SHEAR STRENGTH (Su, kPa): ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, ▼ REMOLD VANE, × PENETROMETER; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%)

UNIT WEIGHT γ kN/m³



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) 1/31/24 12:05:00 PM	1.1 ▽	1.3
2) 1/31/24 12:37:00 PM	-	1
3)	-	-

MEL-GEO 2312415 - RIVER ROAD.GPJ MEL-GEO.GDT 3/8/24

ENCLOSURE NO.:9 (Pg. 1 of 2)

METRIC

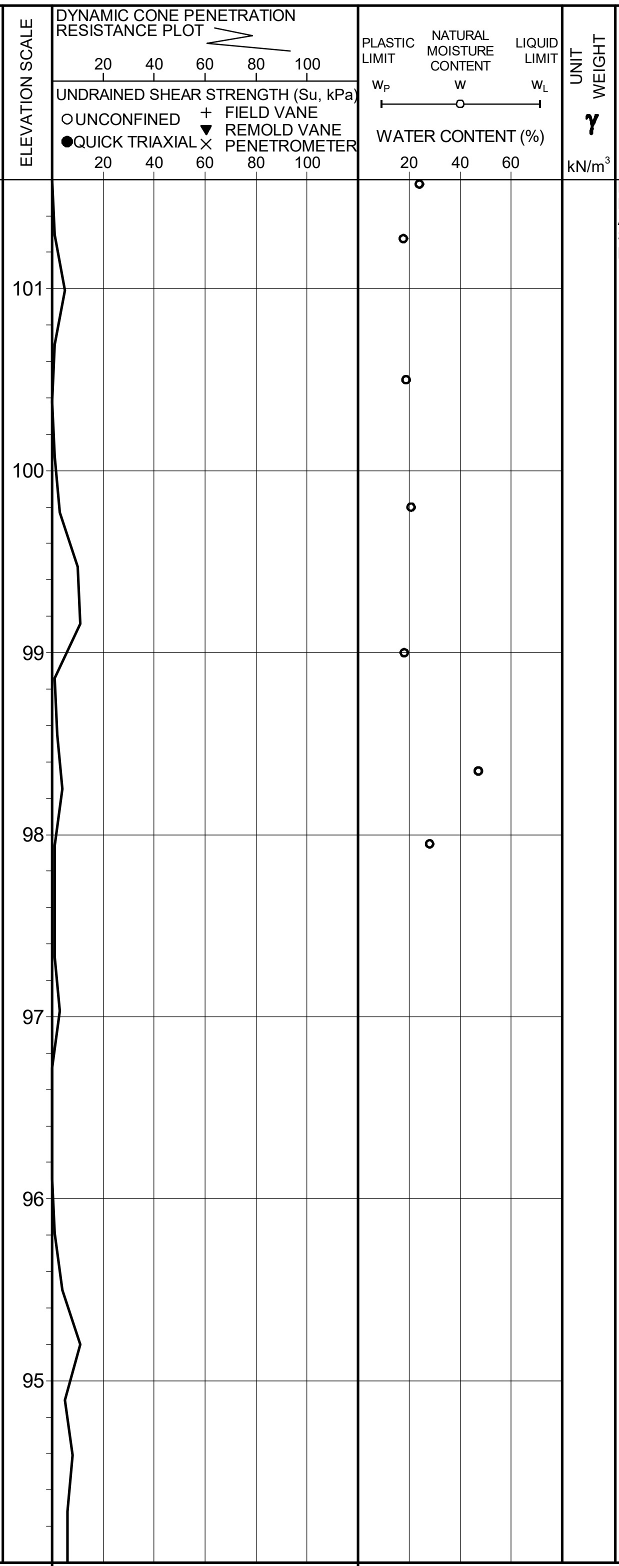
RECORD OF BOREHOLE NO. 7

ENGLOBE

REFERENCE 02312415.000 DATUM TBM LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2 ORIGINATED BY ES
PROJECT GI - 243 River Road, Sault Ste. Marie BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger COMPILED BY DMc
CLIENT Ontario Aboriginal Housing Services DATE (Started) January 31, 2024 DATE (Completed) January 31, 2024 TIME (Completed) 11:15:00 AM CHECKED BY RT

ELEV DEPTH	DESCRIPTION (see Enclosure No. 1)	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
101.6						
[illegible] 0.1	TOPSOIL - 50 mm SILT - some sand, brown, loose	1A 1B	SS	6		Dynamic Cone Penetration Test Advanced 0.9 m South of Original Borehole
100.9 0.8	SILTY SAND - trace gravel, clay, brown, loose	2	SS	5	▽	7 57 (36)
	- wet	3	SS	3	🟊	
	- gravelly	4	SS	14		
98.6 3.1	SILTY CLAY - brown, soft	5A	SS	1		
98.3 3.4	SAND - trace silt, brown, loose	5B				
97.8 3.8	End of Sampling at 3.8 m bgs					

Continued Next Page



COMMENTS

$+^3, \times^3$: Numbers on right refer to Sensitivity
Numbers on left refer to values greater than 100 kPa
$\bigcirc^{3\%}$ STRAIN AT FAILURE

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)		Cave In (m)	
1) 1/31/24 12:44:00 PM	1.3	▽	1.9	🟊
2)	-	▽	-	
3)	-	▼	-	

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

ENCLOSURE NO.:9 (Pg. 2 of 2)

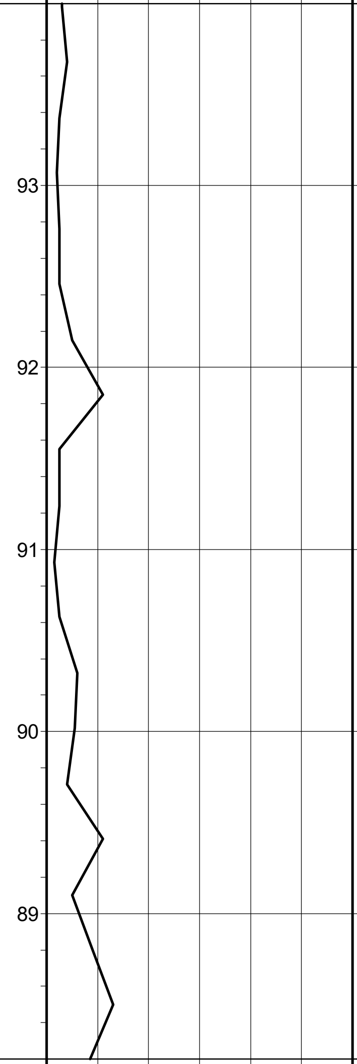
METRIC

RECORD OF BOREHOLE NO. 7

ENGLOBE

REFERENCE 02312415.000 DATUM TBM
LOCATION See Borehole Location Plan; Appendix A, Dwg. No. 2
ORIGINATED BY ES
PROJECT GI - 243 River Road, Sault Ste. Marie
BOREHOLE TYPE Geoprobe 7822DT Drill Rig - Hollow Stem Auger
COMPILED BY DMc
CLIENT Ontario Aboriginal Housing Services
DATE (Started) January 31, 2024
DATE (Completed) January 31, 2024
TIME (Completed) 11:15:00 AM
CHECKED BY RT

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION (see Enclosure No. 1) Continued from Previous Page	STRATA PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 UNDRAINED SHEAR STRENGTH (Su, kPa) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL ▼ REMOLD VANE × PENETROMETER 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)
88.2 13.4	End of Borehole at 13.4 m bgs										

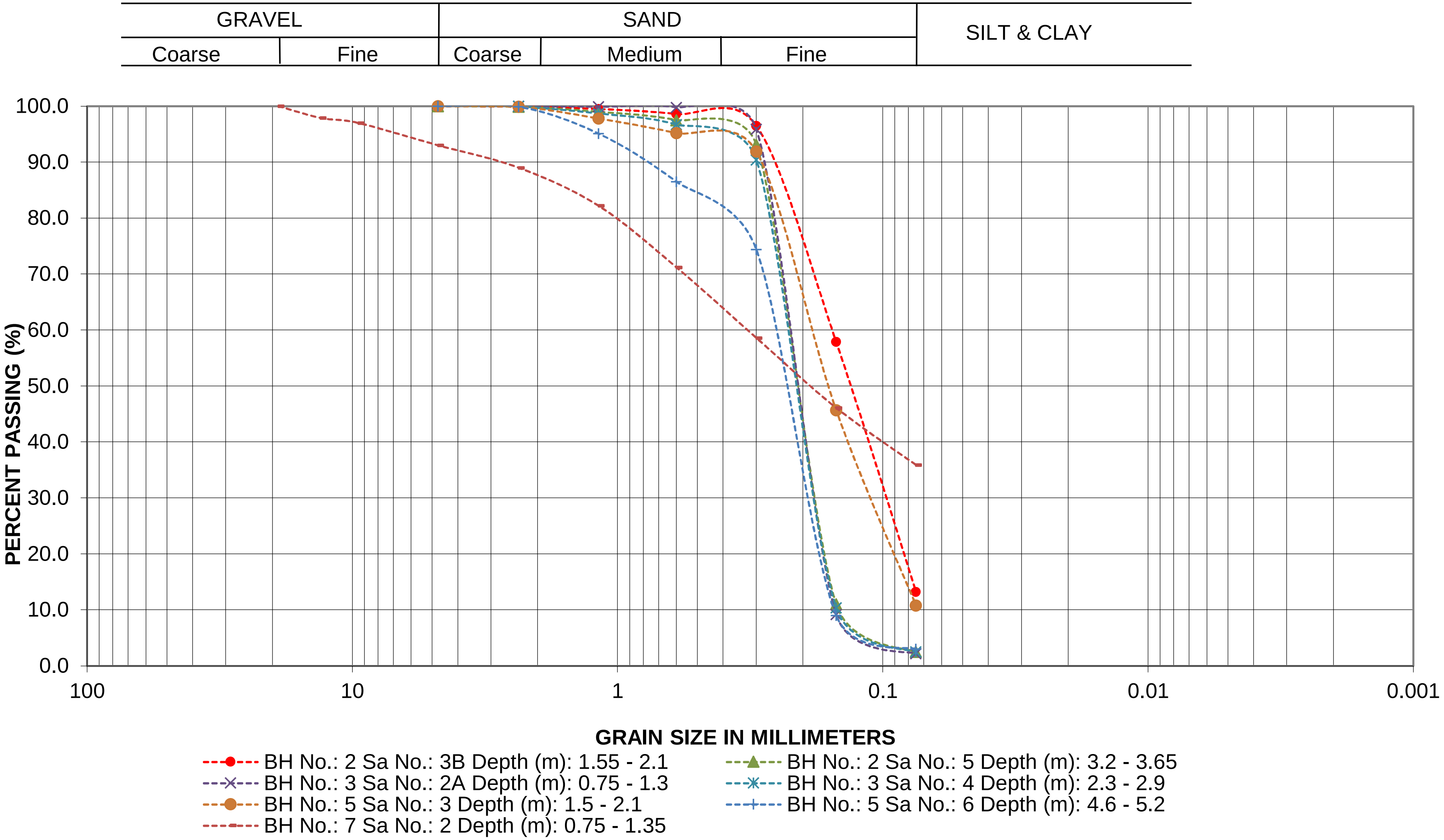


Appendix C
Laboratory Test Results

Lab Data

Reference No.: 02312415
Date: February 2024

GRAIN SIZE ANALYSIS



SANDS to SILTY SANDS

PROJECT: Proposed Housing Development
LOCATION: 243 River Road, Sault Ste Marie, Ontario

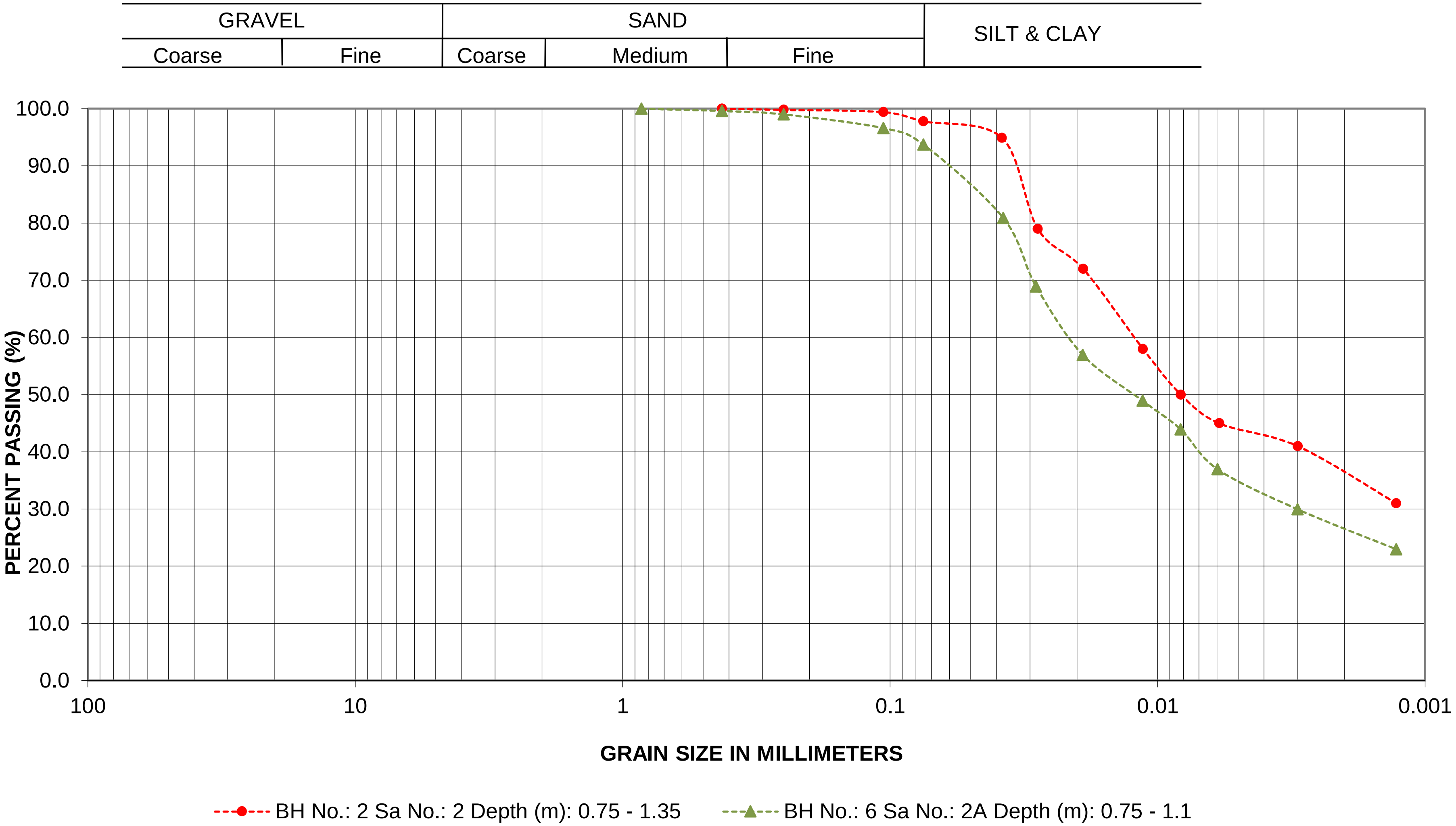
Englobe Corp.

FIGURE L-1

Reference No.: 02312415
Date: February 2024

ENGLOBE

GRAIN SIZE ANALYSIS



SILTY CLAYS

PROJECT: Proposed Housing Development
LOCATION: 243 River Road, Sault Ste Marie, Ontario

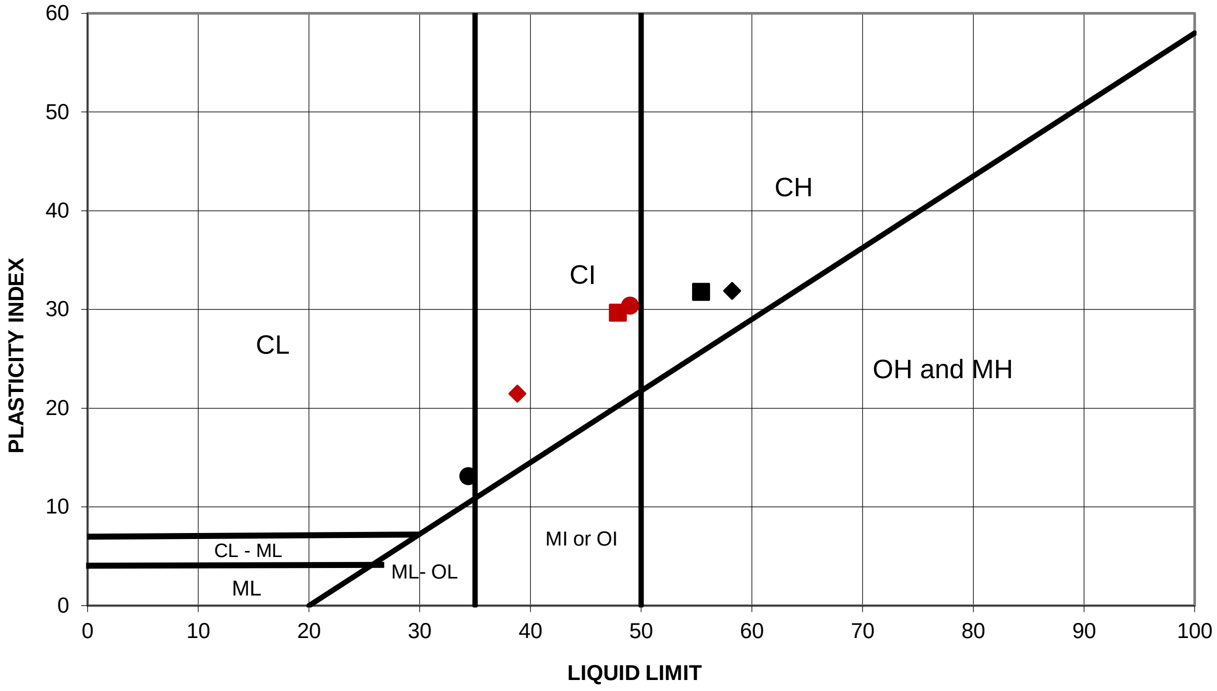
Englobe Corp.

FIGURE L-2

ATTERBERG LIMITS TEST RESULTS

FIGURE L-3

ATTERBERG INDICES



SYMBOL	BH No.	Sa No.	Depth (m)	Plasticity Index	Plastic Limit	Liquid Limit	NMC %
●	2	2	0.75 - 1.35	13.1	21.3	34.4	27.4
■	2	4	2.3 - 2.9	31.8	23.6	55.4	64.4
◆	2	6	4.6 - 5.2	31.9	26.3	58.2	63.6
●	3	7	4.6 - 5.2	30.4	18.6	49.0	38.3
■	5	4A	2.3 - 2.6	29.7	18.2	47.9	57.0
◆	5	8	7.6 - 8.2	21.5	17.3	38.8	41.4

Date: 13-Feb-24 Prepared By: DMc
Project: Proposed New Housing Development, 243 River Road, Sault Ste Marie, Ontario

ENGLOBE CORP.

Appendix D
Photo Essay

Photo Essay

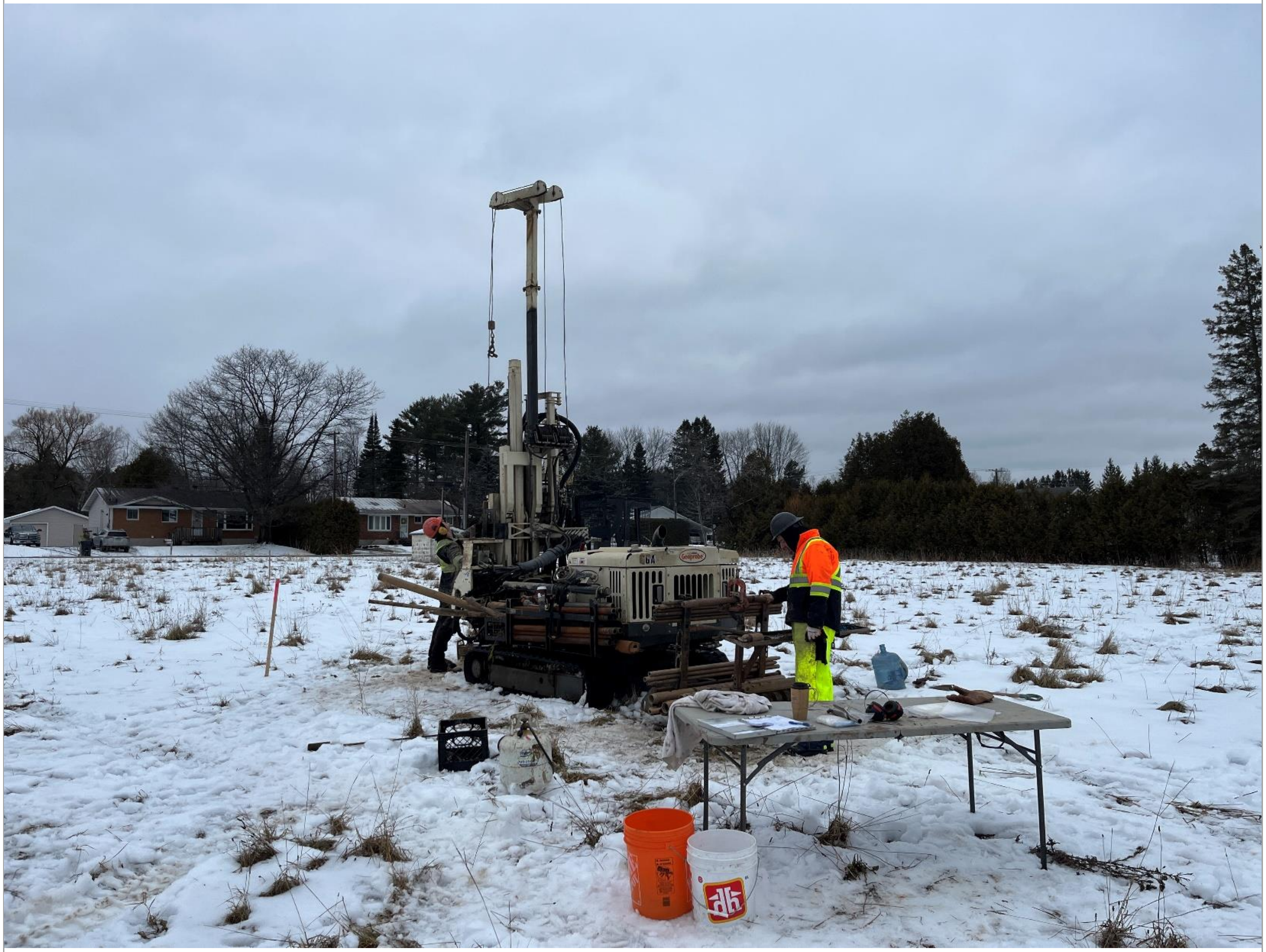
Site Overview	Photo: 1.

Project: GI, 243 River Road, SS Marie	Photos by: Englobe Date: February 2024

Utility Mark-Up Near BH-05	Photo: 2.

Project: GI, 243 River Road, SS Marie	Photos by: Englobe Date: February 2024

Drilling in Progress	Photo: 3.



Project: GI, 243 River Road, SS Marie	Photos by: Englobe Date: February 2024

MW1005	Photo: 4.

Project: GI, 243 River Road, SS Marie	Photos by: Englobe Date: February 2024

Backfilled after Drilling BH-07	Photo: 5.

Project: GI, 243 River Road, SS Marie	Photos by: Englobe Date: February 2024

Appendix E
Seismic Hazard Calculations

Seismic Hazard Values

User requested values

Code edition	NBC 2020
Site designation X_S	X_E
Latitude (°)	46.52
Longitude (°)	-84.251

Please select one of the tabs below.

NBC 2020 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 2020. 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 2020.

See the *Additional Values* tab for additional seismic hazard values, including values for other site designations, periods, and probabilities not defined in the NBC 2020.

NBC 2020 - 2%/50 years (0.000404 per annum) probability

$S_a(0.2, X_E)$	$S_a(0.5, X_E)$	$S_a(1.0, X_E)$	$S_a(2.0, X_E)$	$S_a(5.0, X_E)$	$S_a(10.0, X_E)$	$PGA(X_E)$	$PGV(X_E)$
0.14	0.162	0.1	0.0478	0.0119	0.00372	0.0805	0.0955

The log-log interpolated 2%/50 year $S_a(4.0, X_E)$ value is : **0.0167**

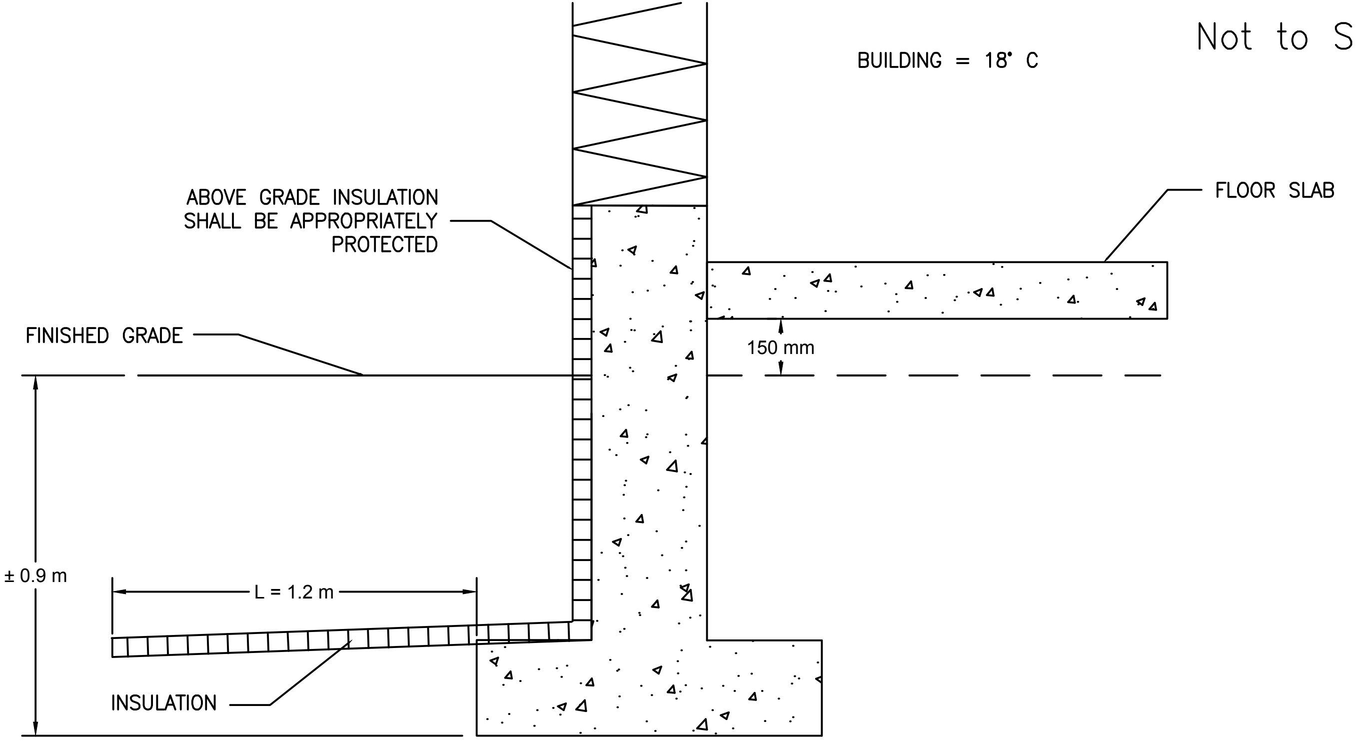
Appendix F
Typical Insulation Details

ENGLOBE

TYPICAL DETAIL
HEATED FOOTING

Englobe

FIGURE No. B
Not to Scale



TYPICAL DETAIL
UNHEATED FOOTING