Ontario Northland

555 Oak Street East
North Bay, Ontario
P1B 8L3

555, rue Oak Est
North Bay (Ontario)
P1B 8L3

Tel: 1-800-363-7512
www.ontarionorthland.ca

June 11, 2026

Addendum No. 03

File Reference Number: RFP 2026 030

Title: North Bay Powerhouse Compressed Air System Upgrades

RE: Clarifications/Questions

QUESTIONS/CLARIFICATIONS:

Item 1: With respect to Item No. 08 of Addendum No. 01: Would ONTC provide the geotechnical report associated with the new enclosure foundations, exterior equipment pads, and related site work?

Answer: ONTC has attached the Geotechnical Investigation and Design Report dated November 9, 2023 to this Addendum No. 03, for reference purposes only and shall not be relied upon for construction.

ONTC also wishes to note that this response is provided in addition to the response issued under Item No. 08 of Addendum No. 01 which remains applicable.

Item 2: Would ONTC provide information on existing door hardware that doors are to match?

Answer: No, ONTC will not be providing specific existing hardware details. However, proponents may propose suitable quality industrial grade hardware. Submissions should reflect industry-standard products appropriate for the intended application.

This Addendum hereby forms part of the RFP.

Regards,

Nicole Laplante
Procurement Contracts Specialist
nicole.laplante@ontarionorthland.ca

Geotechnical Investigation and Design Report

Ontario Northland Transportation Commission

Type of Document:
Report

Project Name:
Paint Shop and Exterior Upgrades
915 McIntyre Street
North Bay, Ontario

Project Number:
SUD-23012250-A0

Prepared By:
Ian MacMillan, P.Eng.
Project Manager, Earth and Environmental Services
EXP
885 Regent Street
Sudbury, Ontario, P3E 5M4
t: +1.705.674.9681
f: +1.705.674.5583

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Further to our Proposal No. 23/134/GP, dated September 8, 2023, and your subsequent authorization to proceed, EXP Services Inc. (EXP) has completed the field investigation and geotechnical engineering evaluation for the proposed upgrades. Our comments and recommendations, based on the results of the field investigation and our understanding of the project scope, are provided in this report.

1. Introduction

It is understood by EXP that upgrades are proposed for Ontario Northland Transportation Commission's (ONTC) Paint Shop Building, as well as various exterior upgrades, located at 915 McIntyre Street in North Bay, Ontario. Specific details of the various upgrades have not been provided to EXP at this time, however, it is understood that various structures on shallow foundations are proposed.

In order to assist with the design of the proposed upgrades, EXP has completed a geotechnical investigation at the site, with the results of the investigation and associated recommendations included within this report.

2. Field Investigation

The field investigation for this project consisted of eight (8) sampled boreholes and one (1) sampled test pit. Of the eight sampled boreholes, five (5) were advanced through the floor slab in the interior of the existing Paint Shop building, two (2) were advanced on the exterior of the Paint Shop building, and the final borehole was advanced near a proposed Sand Tower, south of the Paint Shop building. The test pit was advanced on the exterior of the paint shop, to the north of the building. The boreholes and test pit were advanced on October 18 and 19, 2023 at the approximate locations shown on Dwg. Nos. A-1 and A-2, included in Appendix A. The advancement of the boreholes and test pit was supervised on a full-time basis by a geotechnical representative from EXP. The retained soil samples were logged in the field and then carefully packaged and transported to our laboratory for detailed examination and testing.

The borehole and test pit locations were laid out in the field by EXP at accessible locations, free of buried services. Private buried services, and rebar within the slab-on-grade, were cleared by RG Sutton Inspection Services. The boreholes were advanced using a trailer mounted Dietrich D25 drill rig equipped with 200 mm diameter Hollow Stem Augers (HSA). Bedrock was cored at noted locations using NW casing and NQ core barrels. An excavator for the advanced test pit was provided by ONTC.

The majority of the boreholes and the test pit were advanced to bedrock, or refusal on suspected bedrock, at the depths shown on the attached logs, Figs. B-2 to B-10, in Appendix B. Borehole BH-8 was terminated at 6.7 m depth, without encountering refusal on suspected bedrock. Bedrock was cored at three (3) borehole locations.

Within the boreholes, Standard Penetration Tests (ASTM D1586) were completed at depths noted on the attached borehole logs in Appendix B. The Standard Penetration Test (SPT) "N" values were recorded and used to provide an assessment of the in-situ compactness condition/consistency of the encountered soils. The boreholes were backfilled with auger cuttings and sealed with bentonite. Boreholes through the existing slab-on-grade were patched using non-shrink concrete.

Within the test pit, sample were obtained at each general change in stratigraphy. The test pit was backfilled with excavated soils that were packed in place as best as possible using the bucket of the excavator.

The borehole locations and elevations were determined in the field using and hand-held GPS unit. For the interior boreholes within the Paint Shop, the surface elevation of each borehole is based on the FFE elevation of 201.5 m (661' 3") as noted on the original design drawings for the facility. The locations and elevations of the boreholes should be considered accurate only to the degree implied by the method used for purposes of this geotechnical report. The elevations and locations noted should not be utilized for detailed design purposes.

3. Laboratory Testing

A laboratory testing program was performed on representative soil samples and consisted of moisture content determinations, grain/particle size analyses, and Atterberg Limits tests. Unconfined Compressive Strength (UCS) testing also completed on representative bedrock core samples. The laboratory test results are summarized on the attached logs in Appendix B, with detailed results included in Appendix C.

4. Subsurface Conditions

Details of the soils encountered during the field investigation are summarized on the attached logs in Appendix B. The logs include textural descriptions of the subsoil and indicate the soil boundaries inferred from non-continuous sampling and observations during the field investigation. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. When reading this report, the explanatory notes and definitions provided in Figures B-1A and B-1B in Appendix B should be referenced.

4.1 Paint Shop Interior (BH-1 to BH-5)

Boreholes BH-1 to BH-5 were advanced within the interior of the existing Paint Shop building. Details of the boreholes are summarized on the attached borehole logs, Figs. B-2 to B-6, included in Appendix B. Borehole locations are shown on Dwg. No. A-1, included in Appendix A.

In general, the boreholes encountered the concrete slab overlying sand fill, additional concrete, and bedrock/suspected bedrock.

The concrete slab was cored at the surface of each borehole. Concrete Slab thicknesses varied at each borehole and are summarized on Table 4-1 below.

Table 4-1: Concrete Core Summary – Interior Boreholes

Borehole No.	Approx. Slab Thickness (mm)
BH-1	380
BH-2	410
BH-3	330
BH-4	280
BH-5	305

At Borehole BH-1, additional concrete was encountered at cored at depth. At 510 mm depth, 100 mm of concrete was encountered, and at 900 mm depth, 380 mm of concrete was encountered above the bedrock surface.

Below the concrete at Boreholes BH-2 to BH-5 and between the concrete layers at BH-1 was sand fill. The sand fill was brown, grey, and black in colour, moist to wet, and contained trace to some gravel and trace silt. Trace organics were encountered within the fill below the second concrete layer at BH-1. Uncorrected SPT “N” values within the sand fill ranged from 3 to 26 blows per 300 mm, classifying the fill as very loose to compact in compactness condition. Measured moisture content within the fill ranged from 5.9 to 24.2%.

Bedrock, or auger refusal on suspected bedrock, was encountered at each borehole location. A summary of the bedrock/suspected bedrock depths is included in Table 4-2 below.

Table 4-2: Bedrock/Suspected Bedrock Summary – Interior Boreholes

Borehole No.	Bedrock Depth (cored)	Suspected Bedrock Depth (auger refusal)
BH-1	1.3	--
BH-2	--	1.7
BH-3	--	2.0
BH-4	--	0.8
BH-5	1.7	--

Where bedrock was cored at Boreholes BH-1 and BH-5, core recovery was 100%. Water return was poor to good (0 to 80%) and was red in colour. Rock Quality Designation (RQD) values generally ranged from 68 to 83%, indicating a fair to good quality bedrock. At Borehole BH-5, the upper 0.5 m had an RQD of 0%, indicating that the surface of the bedrock is very poor quality and highly fractured at this location. Unconfined Compressive Strength (UCS) tests completed on representative bedrock core samples indicated very high strengths ranging from 128.3 to 153.6 MPa.

4.2 Paint Shop Exterior (BH-6, BH-7, and TP-1)

Boreholes BH-6 and BH-7, as well as Test Pit TP-1 were advanced at the exterior of the existing Paint Shop building. Details of the boreholes and test pit are summarized on the attached logs, Figs. B-7, B-8, and B-10, included in Appendix B. Borehole locations are shown on Dwg. No. A-2, included in Appendix A.

Borehole BH-6 encountered 100 mm of asphalt at the surface overlying sand and gravel fill that extended to auger refusal on suspected bedrock at 0.9 m depth. The sand and gravel fill was brown to black in colour, moist, and contained trace silt. One SPT performed within the fill resulted in an uncorrected “N” value of 8 blows per 300 mm, classifying the fill as loose in compactness condition. Measured moisture content within the fill ranged from 5.7 to 8.5%.

Borehole BH-7 encountered 50 mm of asphalt at the surface overlying sand fill that extended to bedrock at 0.6 m depth. The sand fill was brown in colour, moist, and contained some gravel and trace silt. Measured moisture content within the fill was 6.3%. Bedrock was cored at BH-7 and core recovery was 100%. Water return was poor to good (40 to 70%) and was red to grey in colour. Rock Quality Designation (RQD) values ranged from 73 to 84%, indicating a fair to good quality bedrock.

Test Pit TP-1 encountered fill materials overlying shallow bedrock. An approximately 100 mm thick layer of sand fill was encountered at the surface of TP-1. The sand fill was brown in colour, moist, and contained some silt and trace gravel. Underlying the sand fill was silty sand fill that extended to bedrock at 0.8 m depth. The silty sand fill was black in colour changing to brown with depth and moist. Trace to some gravel and trace clay was encountered within the silty sand fill. Measured moisture contents within silty sand fill ranged from 17.1 to 22.5%.

4.3 Sand Tower Location (BH-8)

Boreholes BH-8 was advanced near a proposed sand tower location, south of the Paint Shop building. Details of the borehole are summarized on the attached borehole log, Fig. B-9, included in Appendix B. The borehole location is shown on Dwg. No. A-2, included in Appendix A.

In general, Borehole BH-8 encountered sand fill overlying native silty sand, sand, clay and silt, and silty sand till.

Sand fill was encountered at the surface of the borehole and extended to approximately 1.5 m depth. The sand fill was brown and black in colour, moist, and contained some gravel and some silt. Uncorrected SPT "N" values within the sand fill ranged from 11 to 17 blows per 300 mm, classifying the fill as compact in compactness condition. Measured moisture contents within the sand fill ranged from 3.5 to 8.4%.

Underlying the sand fill was native silty sand that extended to approximately 2.3 m depth. The silty sand was brown to grey in colour, wet, and contained trace gravel. One SPT preformed within the silty sand resulted in an uncorrected "N" value of 9 blows per 300 mm, classifying the silty sand as loose in compactness condition. Measured moisture content within the silty sand was 23.8%.

Underlying the silty sand was native sand that extended to approximately 4.6 m depth. The native sand was grey in colour, wet, and contained some silt. Uncorrected SPT "N" values within the silty sand ranged from 13 to 18 blows per 300 mm, classifying the sand as compact in compactness condition. Measured moisture content within the sand ranged from 27.1 to 30.9%.

Underlying the sand was native clay and silt that extended to approximately 6.1 m depth. The clay and silt was grey in colour, wet, and contained trace sand. One SPT performed within the clay and silt resulted in an uncorrected "N" value of 1 blow per 300 mm. A field vane test performed within the clay and silt resulted in an undrained shear strength of 20 kPa. As such, the clay and silt is classified as very soft to soft in consistency. An Atterberg Limits test performed on a representative sample of the clay and silt indicated a Liquid Limit of 45.4%, a Plastic Limit of 22.5%, and a Plasticity Index of 22.9%. As such, the clay and silt has moderate plasticity and is considered moderately compressible. Measured moisture content within the clay and silt was 64.2%

Below the clay and silt was native silty sand till that extended to the borehole termination depth of 6.7 m. The till was grey in colour, wet, and contained some gravel. One SPT performed within the till resulted in an uncorrected "N" value of 32 blows per 300 mm, classifying the till as dense in compactness condition. Measured moisture content within the till was 19.0%.

4.4 Groundwater

Groundwater was not encountered within the boreholes during the short term the holes were left open upon completion. However, samples below 1.5 to 2.1 m at Borehole BH-8 were generally wet, which could infer a groundwater level at or near this depth. At Test Pit TP-1, water was noted to be trickling into the open test pit along the bedrock surface. Seasonal variations in the water table should be anticipated, with higher levels occurring during wet weather conditions (spring thaw and late fall) and lower levels occurring during dry weather conditions.

5. Discussion and Recommendations

Within the existing paint shop or in the vicinity of Boreholes BH-6, BH-7 or Test Pit TP-1, it is recommended to found proposed structures on conventional strip or spread footings bearing directly on bedrock, or engineered fill overlying bedrock, as shallow bedrock was encountered throughout these areas.

Near the proposed sand tower location, compact native cohesionless soils were encountered below 1.5 m depth, however, a moderately compressible, weaker, layer of cohesive clay and silt was encountered between 4.6 to 6.1 m depth. In order not to overly load the weaker clay and silt layer and to minimize potential long-term consolidation settlements, it is recommended to keep lightly loaded foundations as shallow as possible (i.e. < 1m depth from existing grade). In addition, grade raises must be kept to a minimum (i.e. less than 0.3 m) to further prevent long-term consolidation settlements. With shallow foundations of this nature, sufficient earth cover frost protection will not be available and as such, insulation would be required. Note that existing fill materials would need to be removed down to native soils and replaced with new engineered fill below the foundations.

As an alternative to shallow foundations for the proposed sand tower structure, deep foundations consisting of helical piles founded below the cohesive clay and silt within the encountered dense silty sand till can be considered. Helical piles founded within the till can support larger bearing pressures than shallow foundations in this area without concern for long-term consolidation settlements.

Foundation recommendations are provided in the following sections for the various foundation types recommended above including:

- Conventional Strip or Spread Footings on Bedrock
- Conventional Strip or Spread Footings on Engineered Fill Overlying Bedrock
- Shallow Foundations on Engineered Fill Overlying Native Soils
- Helical Pile Foundations

5.1 Conventional Strip or Spread Footings on Bedrock

As noted previously, proposed structures within the paint shop building or within the vicinity of Boreholes BH-6, BH-7 or TP-1 can be founded on conventional strip or spread footings bearing directly on sound bedrock.

Shallow foundations bearing on bedrock can be designed with a factored geotechnical resistance at Ultimate Limit States (ULS) of 2.0 MPa. A geotechnical resistance factor of 0.5 has been applied to determine this value. Serviceability Limit States (SLS) design does not apply for footings bearing directly on bedrock as failure of the concrete would occur before unacceptable settlement of the foundation. For footings bearing directly on bedrock, settlements will be negligible.

The recommended geotechnical resistance above assumes that all foundation concrete is established on sound unweathered rock, which has been cleaned of all loose debris and rock shatter using air hose or water jetting procedures. Footings should be placed on fairly level bedrock (i.e. sloping less than 10° from the horizontal). In some instances, lightly loaded spread footings may be placed on bedrock sloping up to 25° to 30° from the horizontal as long as rock dowels are incorporated into the design to ensure sufficient resistance against sliding. As an alternative to levelling the bedrock surface by mechanical or blasting techniques, where the bedrock is irregular with erratic changes in profile, ledges, crevices, etc., the footing beds may be levelled by benching over these areas with mass concrete (min. 20 MPa compressive strength), anchored into the bedrock where the overall slope of the bedrock across the base of the foundation exceeds 10°. Typically, this decision is made on-site, depending on site specific bedrock conditions.

All bedrock surfaces must be reviewed by EXP prior to pouring foundation concrete. This is necessary to verify the assumed foundation bearing conditions and review the foundation construction procedures, bedrock slope, etc.

Strip and spread footing widths must comply with the Ontario Building Code minimum requirements.

5.1.1 Rock Dowels and Anchors

If rock anchors or dowels are required, the structural engineer normally designs the length and diameter of the steel anchors/dowels for footings, based on the type of bedrock and its strength parameters. Rock anchors can also be used to support the utility poles (guy anchors).

For bedrock in the North Bay area, failure typically occurs between the dowel and the grout, or between the grout and the rock, and not from a quasi-conical rock mass failure, provided sufficient dowel bond lengths have been designed. Empirical methods of analysis, such as pull-out tests have shown that the bond developed between the grout and the dowel are typically twice that of the bond developed between the grout and the bedrock. Therefore, the design analysis should be based on failure occurring between the grout and the bedrock interface. For straight-shafted dowels, the anchor force, which can be developed, is dependent on the ultimate bond stress of the bedrock or the grout material.

The ultimate bond stress is typically taken as 10% of the unconfined compressive strength of the bedrock or the compressive strength of the grout material, whichever is less, but not more than 3.0 MPa. As unconfined compressive strengths are quite high for the encountered bedrock, 3.0 MPa should be used for the ultimate bond stress assuming a minimum 30 MPa grout is used. The allowable bond stress, “τ_b” taken between the rock and the grout is normally 50% or less of the ultimate bond stress, (i.e. Safety Factor of 2.0 for competent rock in the Creighton area).

The required bond length (L, in metres) for the anchor is a function of the core hole diameter (d), and can be calculated as follows:

$$\mathbf{L = P / (\pi \times d \times \tau_b)}$$

where

P = working capacity of anchor (kN)
τ_b = working bond stress
d = core hole diameter (m)

The upper 300 mm of the bedrock is not normally considered part of the bond length, since this area is usually weathered/fractured, and as a result does not usually develop the ultimate bond stress assumed in the above calculations.

During construction, pullout tests equal to the design loads must be performed by a qualified geotechnical engineer to confirm the strength of the anchors. This work can be performed on a representative number of anchors by EXP.

5.2 Conventional Strip or Spread Footings on Engineered Fill Overlying Bedrock

Proposed structures within the paint shop building or within the vicinity of Boreholes BH-6, BH-7 or TP-1 can be founded on conventional strip or spread footings bearing on engineered fill overlying bedrock.

Conventional strip or spread footings, or thickened edge slab-on-grade foundations, bearing on the engineered fill overlying bedrock may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 385 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 250 kPa, subject to inspection during construction. A geotechnical resistance factor of 0.5 was utilized for the ULS values. With a geotechnical reaction at SLS of 250 kPa, anticipated settlements would be less than the typically acceptable level of 25 mm total.

Prior to engineered fill placement, all in-situ fill, overburden soils, and any other deleterious materials are to be removed down to bedrock. All required up fill beneath the foundations is to consist of a Granular “B” Type II in accordance with Ontario Provincial Standards Specifications (OPSS) 1010. A final 300 mm thick layer of Granular “A” (OPSS 1010) should be placed directly below the foundation. All fill material should be placed in maximum 150 mm thick lifts and be compacted to 100% of the Standard Proctor Maximum Dry Density (SPMDD) within 1.5% of the optimum moisture content. The minimum required thickness of the engineered fill pad is 300 mm.

The engineered fill pad is to extend laterally a minimum of 300 mm beyond all edges of the foundations and then slope down at a slope of one horizontal to one vertical (1H:1V) to the bedrock surface. Engineered fill placement is to be completed under the full-time supervision of EXP to ensure that the recommendations contained herein are met.

All bedrock surfaces must be reviewed EXP prior to placing engineered fill. This is necessary to verify the assumed foundation bearing conditions and review the foundation construction procedures, bedrock slope, etc. Upon exposing the bedrock, if large slopes are observed along the edges of the engineered fill pad or under the proposed structures, it may be required to bench the sloping bedrock to ensure the stability of the fill.

Foundations which are to be placed at different elevations on engineered fill or near service trenches should be located such that the footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of a lower foundation or bottom of a service trench, as indicated on Figure 5-1.

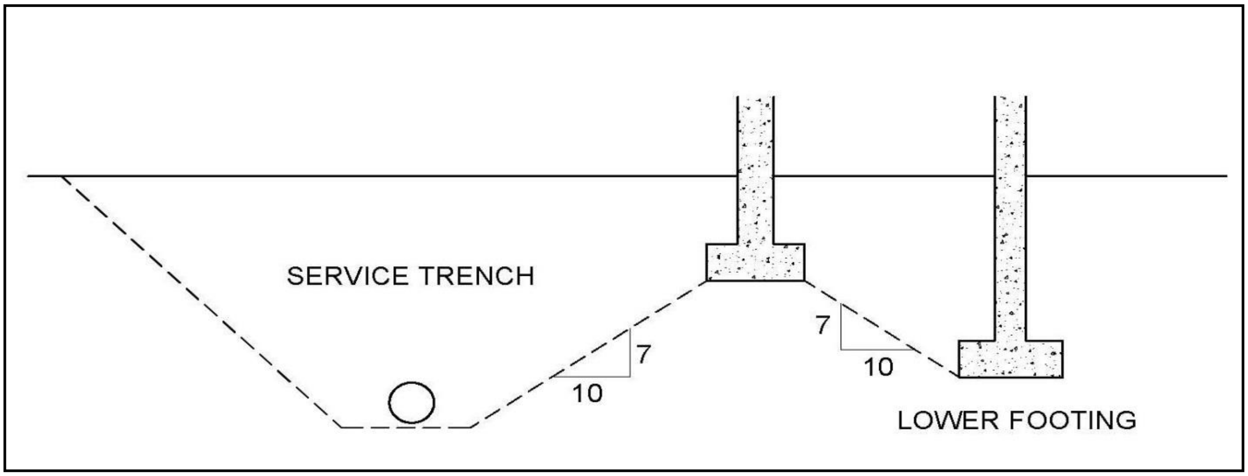


Figure 5-1: Footings near Service Trenches or at Different Elevations

Strip and spread footing widths must comply with the Ontario Building Code minimum requirements.

5.3 Shallow Foundations on Engineered Fill Overlying Native Soils

For the proposed sand tower foundations near Borehole BH-8, shallow strip or spread footings can be utilized. As noted, previously it is not recommended to found the proposed foundations deeper than 1.0 m depth below existing grade in order to prevent significant loading on the underlying moderately compressible cohesive clay and silt soils and ultimately mitigate long-term consolidation settlements. At this depth, sufficient earth cover frost protection would not be available, and insulation as outlined in Section 5.5 below will be required.

Prior to placing any engineered fill, all in-situ fill, organics, or other deleterious materials must be removed down to native soils. Excavations upwards of 1.5 m (or deeper) are anticipated based on encountered fill depths within Borehole BH-8. The exposed subgrade must be proof rolled to identify any soft or unstable areas. The exposed subgrade and proof rolling are to be inspected by a representative from EXP prior to placing fill materials or concrete. Any soft or loose areas encountered below the foundation locations or any areas that are subject to softening/loosening when exposed to water and construction activities should be excavated down to a firm subgrade and replaced with Granular “A” or Granular “B” Type II in accordance with Ontario Provincial Standards and Specifications (OPSS) 1010. If wet soil conditions are present during construction, a non-woven geotextile separator (Terrafix 270R or equivalent) is to be used between the subgrade soils and any engineered fill.

Required up fill beneath the foundations is to consist of a Granular “B” Type II in accordance with Ontario Provincial Standards Specifications (OPSS) 1010. A final 300 mm thick layer of Granular “A” (OPSS 1010) should be placed directly below the foundation. All fill material should be placed in maximum 150 mm thick lifts and be compacted to 100% of the Standard Proctor Maximum Dry Density (SPMDD) within 1.5% of the optimum moisture content.

In order to prevent long-term consolidation settlements within the underlying moderately compressive clay and silt soils, grade raises in the vicinity of the proposed foundations shall not exceed 0.3 m above existing grades.

Conventional strip or spread foundations bearing on engineered fill overlying undisturbed native soils can be designed with a factored geotechnical resistance at Ultimate Limit States (ULS) of 115 kPa, calculated using a geotechnical resistance factor of 0.5. A bearing pressure at Serviceability Limit States (SLS) of 75 kPa may be used. Footings designed with the recommendations contained herein are expected to settle less than 25 mm total and 20 mm differential.

Engineered fill placed below the foundations is to extend horizontally a minimum of 0.5 m beyond the edges of the foundation and slope down to the native soils at 1H:1V to ensure the foundation loads are properly transferred to the underlying subgrade. Engineered fill placement and compaction below foundations is to be continuously monitored on a full-time basis by a qualified geotechnical representative from EXP.

The location of any foundation on an engineered soil pad is critical and certification by a qualified surveyor that the foundations are within the stipulated boundaries is mandatory. Since layout stakes are often damaged or removed during fill placement, offset stakes must be installed and maintained by the surveyors during the course of fill placement so that the contractor and engineering staff are continually aware of where the engineered fill limits lie.

Foundations which are to be placed at different elevations in soils or near service trenches should be located such that the footings are set below a line drawn up at 10 horizontal to 7 vertical from the near edge of a lower foundation or bottom of a service trench, as indicated on Figure 5-1 noted previously.

Strip and spread footing widths must comply with minimum Code requirements.

5.4 Helical Pile Foundations

For the proposed sand tower near Borehole BH-8, helical pile foundations can be utilized it larger bearing pressures are required that those recommended for shallow foundations as detailed above in Section 5.3.

The helices will need to be established within the dense silty sand till encountered below 6.1 m depth. Helices must be established below this minimum depth of 6.1 m, even if minimum installation torque values are achieved above this depth.

As helical systems are typically proprietary, the recommendations outlined below are intended to assist the designers in developing the foundation system. Note that any other design considerations including, but not limited to, pile spacing, depth, size, etc. are the responsibility of the designer. For installation, EXP recommends utilizing a contractor who specializes in helical pile foundations, particularly one with experience in Northern Ontario. It is also recommended that Chance Helical Piles be used due to their superior capacity and corrosive protection.

Helical piles must be designed as end bearing and the friction from any surficial fills soils must be ignored. The helical pile capacity can be calculated using the following equation from the Canadian Foundation Engineering Manual. Note that a geotechnical resistance factor of 0.4 for piles in compression and 0.3 for uplift must be applied to the calculated capacities.

$$Q_h = A_h(s_u N_c + \gamma D_h N_q + 0.5\gamma B N_\gamma)$$

Where,

Q_h = Individual helix bearing capacity
A_h = Projected helix area
S_u = Undrained shear strength
γ = Unit weight of the soil
D_h = Depth to helical bearing plate
B = Diameter of helical plate
$N_c = (N_q - 1) \cot \phi$
$N_q = e^{(\pi \tan \phi)} \tan^2(45 + 0.5\phi)$
$N_\gamma = 0.0663\ e^{0.1623\phi}$

The above capacity equation assumes that helices are vertically spaced at a minimum distance of three (3) times the largest helix diameter in order to avoid overlapping soil stresses.

Based on the soils encountered on site and the recommended minimum installation depth, for preliminary design considerations, a typical SS175 Chance Helical pile would be capable of supporting a compression capacity of 370 kN SLS and 500 kN ULS. It is always recommended that a load testing program (compression and tension) be performed on several of the helical piles to verify that final design capacities are being achieved. Note that preliminary capacities provided are for the specific Chane Helical piles as noted.

5.5 Backfill Recommendations

All imported backfill material used to backfill foundations or grade beams should consist of Granular “B” Type I or Granular “B” Type II (OPSS 1010) material, with a maximum aggregate size not exceeding 120 mm. The Granular “B” material must be placed in lifts no greater than 150 mm in thickness and must be compacted to 98% of the SPMDD. Care must be taken to ensure damage to the foundation walls does not occur.

5.6 Frost Considerations

The freezing index in the North Bay area is approximately 1,175 C degree-days. There is potential for up to 2.0 m of frost penetration to occur over the winter months in unprotected, unheated areas and 1.6 m for heated structures. For a structure to be considered heated, a minimum interior temperature of 18° C must be maintained throughout the year.

Foundations for unheated structures should be provided with a minimum of 2.0 m of earth cover frost protection and heated structures should be provided with 1.6 m of earth cover frost protection. Where sufficient earth cover frost protection is not provided for the foundations, insulation would be required. Insulation should consist of rigid extruded polystyrene, have a minimum compressive strength of 275 kPa, and an R-Value of 5 for every 25.4 mm of thickness, (i.e. Styrofoam HIGHLOAD 40). Any exposed insulation is to be protected against sunlight and physical damage. A rough estimate for cost evaluation purposes can be made by assuming that 25.4 mm of rigid insulation designed for below grade installation is equivalent to 300 mm of soil cover. Note that insulation for heated structures should be placed both horizontally and vertically along the outside edge of the foundation. Insulation for unheated structures must extend below the entire foundation. Higher compressive strength insulation (i.e. Styrofoam HIGHLOAD 60 or 100, etc.) may be required if insulation extends below foundations, depending on foundation loading conditions.

Detailed insulation recommendations can be provided by EXP, if necessary, once the final foundation designs have been determined.

5.7 Lateral Earth Pressure

Any foundations or retaining structures should be designed to resist lateral earth pressure. The expression for calculating lateral earth pressure "p" at any depth "h" is given by the following:

$$\mathbf{p = K(\gamma h + q) + \gamma_w h_w}$$

where
p = Lateral earth pressure (kPa)
K = Coefficient of earth pressure
γ = Unit weight of backfill (kN/m^3)
γ_w = Unit weight of water (kN/m^3)
h = Depth to point of interest (m)
h_w = Depth of water above point of interest (m)
q = Surcharge load acting adjacent to the wall at the ground surface (kPa)

Table 5-1, below, list various earth pressure properties for given materials.

Table 5-1: Earth Pressure Properties

Material	Friction Angle ø´ (unfactored)	Coefficient of Active Earth Pressure (k_a)	Coefficient of Passive Earth Pressure (k_p)	Coefficient of Earth Pressure at Rest (k_o)	Unit Weight γ (kN/m^3)
Granular "A"	38°	0.24	4.2	0.38	22
Granular "B" Type I	35°	0.27	3.7	0.43	21
Granular "B" Type II	38°	0.24	4.2	0.38	21

Note: Values given for horizontal earth pressures are for horizontal backfill. For sloping backfill, the design requirements outlined in the Canadian Foundation Engineering Manual should be used.

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effects of compaction surcharge should be taken into account in the calculations of active and at rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

5.8 Site Classification for Seismic Response

The Site Classification for Seismic Response has been estimated based on the boreholes advanced at the site. As the Site Classification for Seismic Response is based on soil conditions in the upper 30 m, assumptions were made by EXP for the soil conditions below the borehole termination depths.

Based on EXP's assumptions, the site is classified as Site Class E where foundations are founded on native soil as per the OBC clause 4.1.8.4, Site Properties and Table 4.1.8.4 A, Site Classification for Seismic Response.

For foundations on bedrock or shallow engineered fill overlying bedrock, Site Class C can be used. With foundations on bedrock, or a relatively thin layer of engineered fill overlying bedrock, there is a potential for using the hard rock or rock categories, Site Class "A" or "B". However, in accordance with Commentary J, Bullet 96 of the NBCC (2015), classifying a site as Site Class "A" or "B" requires that the shear wave velocity be measured. As shear wave velocity measurements were not completed for this site, these categories should not be utilized.

These earthquake/seismic design parameters should be reviewed in detail by the structural engineer and incorporated into the design as required. As this site class is based on an assumption of the soil conditions, the site class may not be sufficient, and it may result in an overdesign of the structure.

If a precise Site Classification is required, or if shear wave velocity measurements are required, EXP can provide a quote to perform the necessary testing.

5.9 Excavations

The in-situ fill and native soils may be classified as Type 3 soils for excavations terminating above the groundwater level in conformance with the Ontario Occupational Health and Safety Act (OHSA). Excavation side slopes in Type 3 soils should remain stable at a slope of 1H:1V. The need to excavate flatter side slopes if excessively wet or soft/loose materials or concentrated seepage zone are encountered, should not be overlooked. Water (i.e. surface water runoff) should not be permitted to enter and/or pond within the construction area.

Excavations must be completed so as not to undermine the foundations/slabs of existing structures. If the existing structures may be undermined by proposed excavations, temporary shoring will be required to support the existing structure. Temporary shoring design will be the responsibility of the Contractor.

All excavations must be completed in accordance with the most recent regulations in the Ontario Occupational Health and Safety Act. The contractor should be aware that slope height, slope inclination, or excavation depths, should in no case, exceed those specified in local, provincial, or federal safety regulations. Such regulations are strictly enforced and, if not followed, the owner, the contractor or earthwork or utility subcontractor could be liable for substantial penalties.

It is important to note that soils and groundwater conditions encountered in the construction excavations may vary significantly across the site. Our preliminary soil classifications are based solely on the materials encountered in widely spaced explorations. The contractor should verify that similar conditions exist throughout the proposed area of excavation prior to construction. If different subsurface conditions are encountered at the time of construction, we recommend that EXP be contacted immediately to evaluate the conditions encountered.

5.10 Dewatering

Groundwater was not encountered within the boreholes upon completion. As such, significant dewatering is not anticipated for excavations less than 2 m depth. Any potential perched groundwater should be possible to remove using conventional construction pumps.

Dewatering requirements will be governed by the time of the year the construction is performed. It is the responsibility of the Contractor to propose a suitable dewatering system based on the time of construction and groundwater levels. The method used should not undermine any adjacent structures or buried services. The dewatering method is the responsibility of the Contractor and the Contractor should submit his proposal to the Prime Consultant for review and approval prior to construction.

6. Construction Constraints Under Cold Weather Conditions

For all construction activities at this site, the following applies:

- During excavations, all subgrade soils must be maintained at a minimum temperature of 5° C.
- No granular material may be placed under frozen conditions, with all fill material maintained at a minimum temperature of 5° C prior to and during installation. If granular fill is to be placed in freezing conditions, the granular fill must be restricted to Granular “B” Type II material. Since Granular “B” Type II has a larger aggregate size, care should be taken to prevent point loading on the underside of the concrete.
- Soils and granular fill material that are in direct contact with fresh concrete must be at a minimum temperature of 5° C prior to pouring the concrete and must be free of snow and ice fragments.
- All granular fill, prior to placement of concrete, must be reviewed by this office to ensure that it is free of frost, buried ice and snow.
- All reinforcing steel in the concrete forms must be free of ice and snow, and must be maintained at a minimum temperature of 5° C.
- During the placement of concrete in cold weather conditions, a field cured cylinder should be placed beside the heated form for a period of 6 days. The field cured cylinder should be returned to a designated laboratory on the sixth day for 7-day compressive strength testing.
- All heated and tarped areas should be monitored for temperature using a max/min thermometer.
- All concrete is to have a minimum of 6% to 8% air entrainment to prevent cracking and shall be maintained at a minimum temperature of 10° C for a period of 4 to 7 days.

The 6% to 8% air entrained concrete during cold weather placement is to prevent significant strength loss of concrete as a result of freezing and thawing. The air entrainment will provide the capacity to absorb stresses during freeze/thaw action.

7. Construction Quality Control

Construction quality control of the “earthworks” should be provided throughout the project by a representative of EXP to verify all design assumptions, recommendations and confirmation of the subsurface soil conditions. This includes inspection of the excavation and subgrade prior to the placement of any structural fill and foundations, to ensure that any and all deleterious materials have been removed and to ensure that the actual conditions are not markedly different from those on which the recommendations made herein are based. Compaction control of structural fill is also recommended as standard practice, as is sampling and testing of aggregates and concrete.

8. Design Review

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the design team responsible for the project. If there are any changes, such as relocation of any structures or other features which may affect our analysis, the information obtained during this investigation may be inadequate and additional field work and reporting may be required.

EXP Services Inc. should be retained to review the final design and specifications to confirm that we are in general agreement with the assumptions on which our recommendations are based. If not accorded the privilege of making this review, EXP Services Inc. will assume no responsibility for interpretation of the recommendations in this report.

9. Limitations

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered that differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations.

Whereas this investigation has estimated the groundwater level at the time of the fieldwork, and commented on general construction problems, the presence of conditions, which would be difficult to establish from our test holes, may affect the type and nature of dewatering procedures which should be used in practice. These conditions include local and seasonal fluctuations in the groundwater table, erratic changes in the soil profile between the tests, and thin layers of soil with large or small permeabilities compared with the general soil mass, etc.

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

The investigation and comments are necessarily ongoing as new information of underground conditions becomes available. For example, more specific information is available with respect to in-situ subsurface conditions between test locations once construction is underway. Subsurface soil interpretation between test holes, as well as the recommendations of this report, should be verified through field inspections provided by EXP to validate the current information for use during the construction stage.

Virtually no scope of work, no matter how exhaustive, can identify all contaminants or all conditions above or below ground. For example, conditions elsewhere on the property may differ from those encountered, and conditions may change with time. Therefore, no warranty is provided that the entire site condition is represented by those identified at specific borehole locations.

This report in no way reflects any on-site environmental considerations.

10. Closure

We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

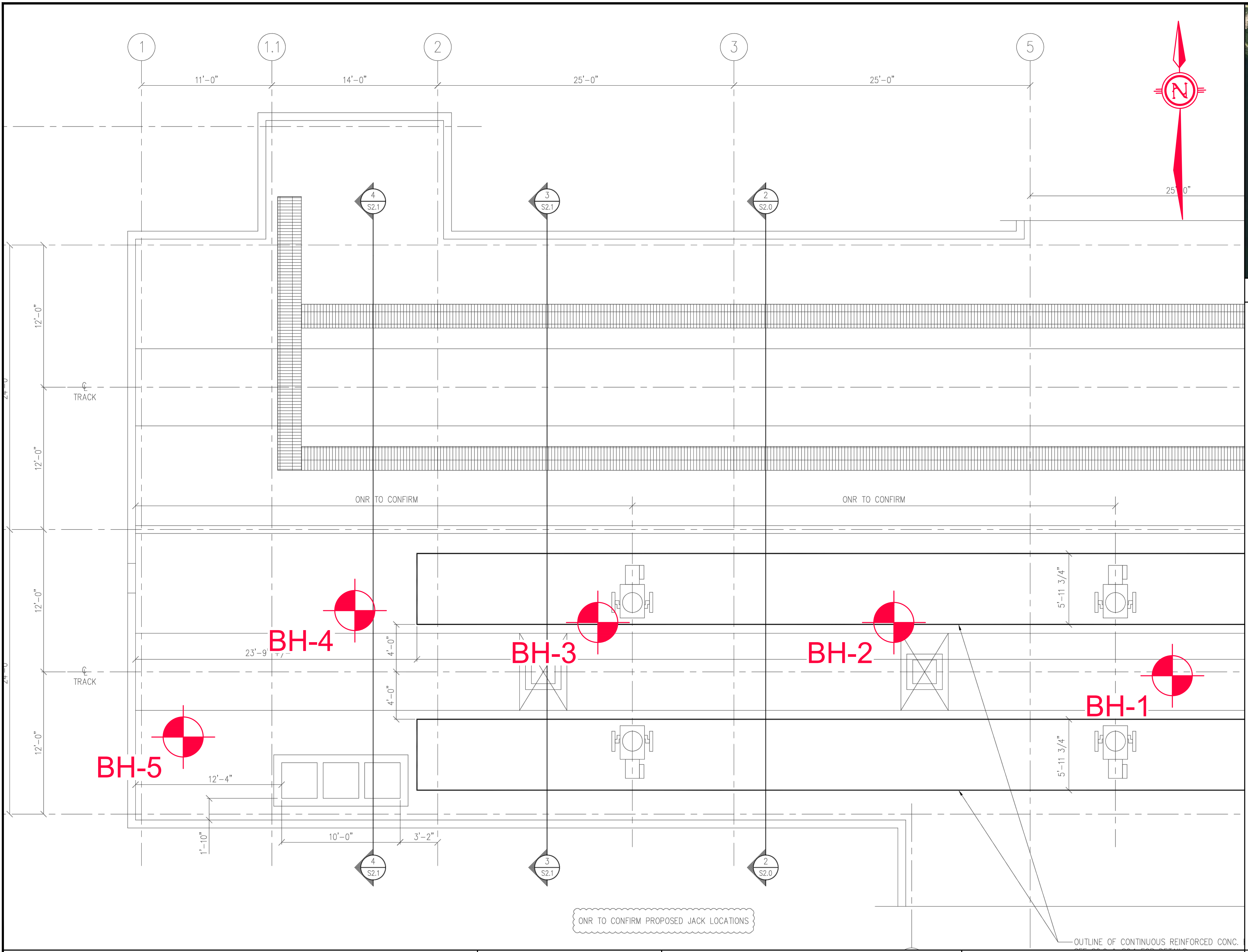
EXP Services Inc.

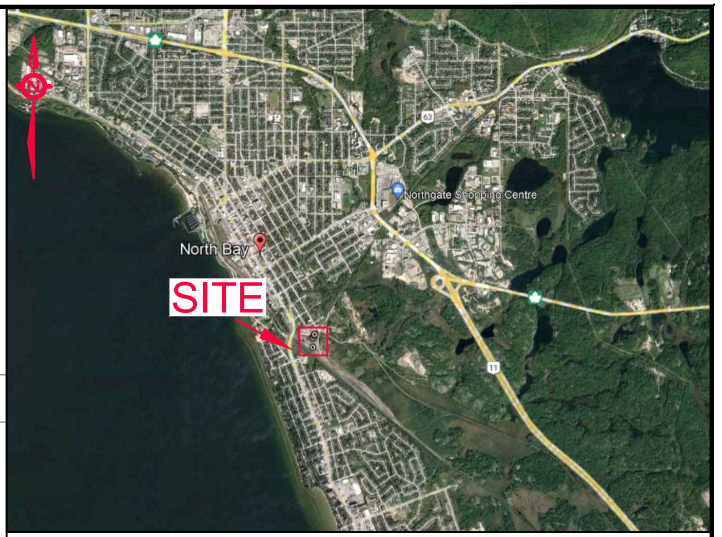
LICENSED PROFESSIONAL ENGINEER
I. B. MACMILLAN
100157509
PROVINCE OF ONTARIO

Ian MacMillan, P.Eng.
Project Manager, Earth & Environmental Services
Northeastern Ontario Northeastern Ontario

Yves Beauparlant, P.Eng.
Manager, Earth & Environmental Services
Northeastern Ontario

Appendix A – Drawings





KEYPLAN - N.T.S.

LEGEND

⊕ EXP BOREHOLE

– NOTES –

1) BH–1–5 were drilling inside the ONTC Paint Shop, therefore, they are not shown on the dwg.

2) The boundaries and soil types have been established only at Test Hole locations. Between Test Holes, they are assumed and may be subject to considerable error.

3) Do not use Test Hole elevations for design purposes.

4) Soil samples will be retained in storage for 3 month and then destroyed unless client advises that an extended time period is required.

5) Quantities should not be established from the information provided at the Test Hole locations.

6) This drawing forms part of the report, project number as referenced, and should be used only in conjunction with this report.

EXP Services Inc.
Sudbury Branch
t: +1.705.674.4401 | f: +1.705.674.5583
885 Regent Street
Sudbury, ON P3E 5M4
Canada

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REVISIONS

No.	DESCRIPTION	DATE

CLIENT
ONTC

PROJECT
ONTC PAINT SHOP AND EXTERIOR
UPGRADES, NORTH BAY, ON

PROJECT NO.
SUD-23012250-A0

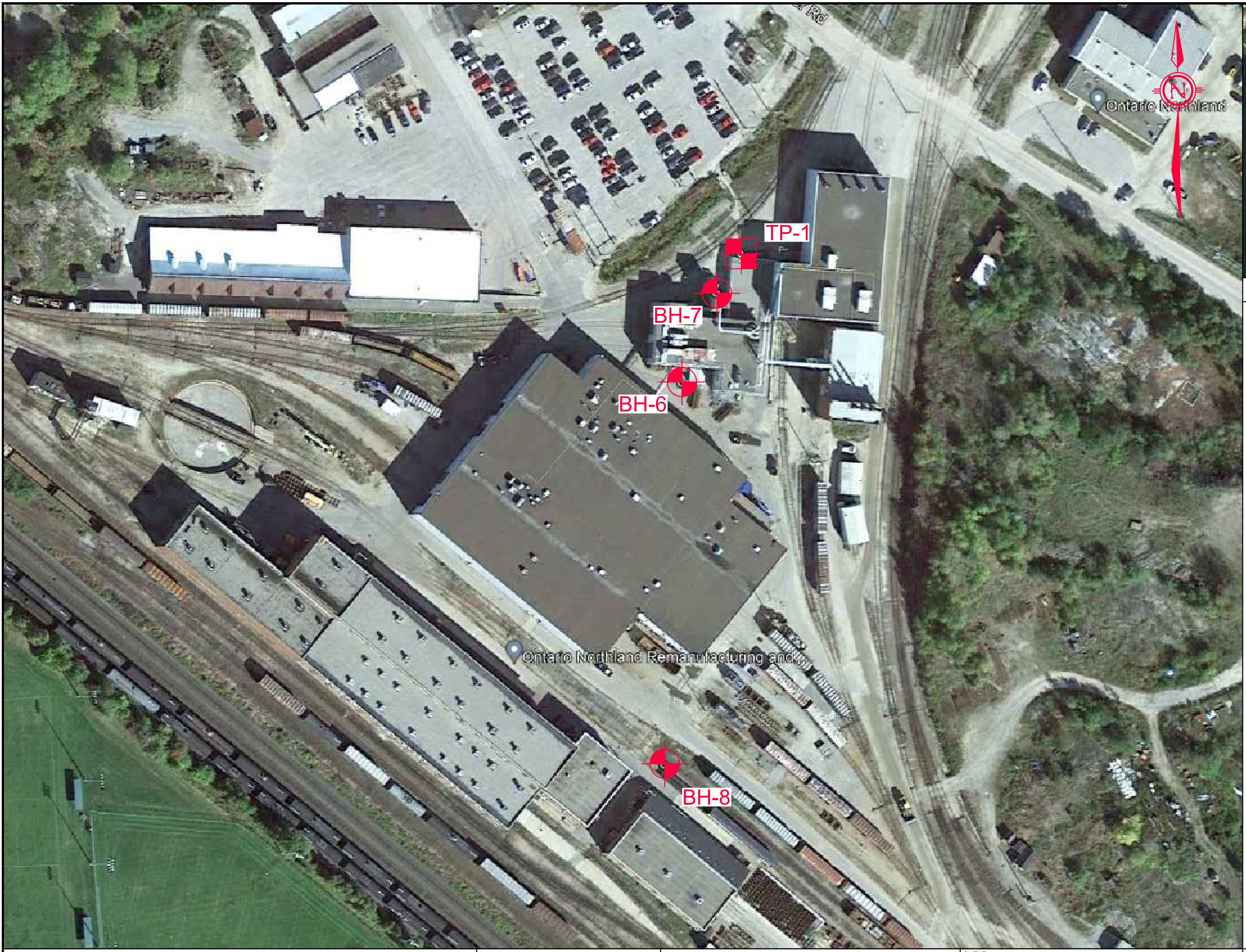
TITLE:
INTERIOR BOREHOLE LOCATION SKETCH PAINT SHOP

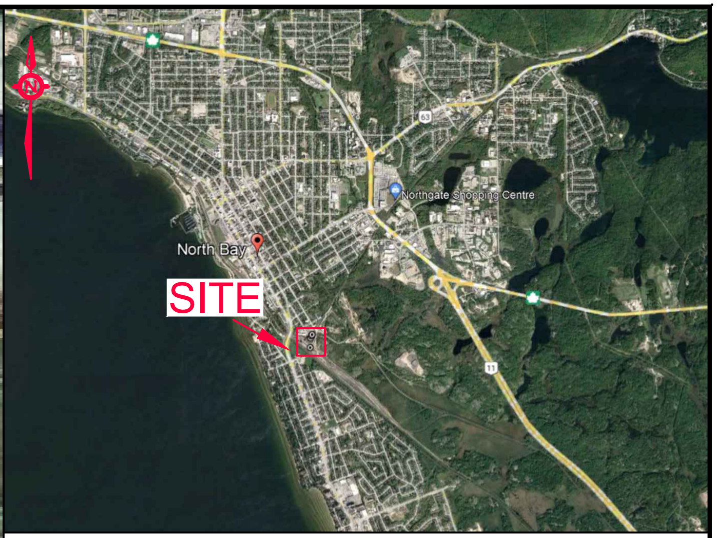
DATE
NOVEMBER 2023

SCALE:
NTS

DWG NO.
A-1

Nov 10, 2023 – 9:09am \\exp\data\SUD\SUD–23012250–A0\60 Execution\65 Drawings\SUD–23012250–A0 – BOREHOLE LOCATION PLAN.dwg





KEYPLAN - N.T.S.

LEGEND

⊕ EXP BOREHOLE

▣ EXP TEST PIT

– NOTES –

1) BH–1–5 were drilling inside the ONTC Paint Shop, therefore, they are not shown on the dwg.

2) The boundaries and soil types have been established only at Test Hole locations. Between Test Holes, they are assumed and may be subject to considerable error.

3) Do not use Test Hole elevations for design purposes.

4) Soil samples will be retained in storage for 3 month and then destroyed unless client advises that an extended time period is required.

5) Quantities should not be established from the information provided at the Test Hole locations.

6) This drawing forms part of the report, project number as referenced, and should be used only in conjunction with this report.

EXP Services Inc.
Sudbury Branch
t: +1.705.674.4401 | f: +1.705.674.5583
885 Regent Street
Sudbury, ON P3E 5M4
Canada

exp.

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REVISIONS

No.	DESCRIPTION	DATE

CLIENT
ONTC

PROJECT
ONTC PAINT SHOP AND EXTERIOR UPGRADES, NORTH BAY, ON

PROJECT NO.
SUD-23012250-A0

TITLE:
EXTERIOR BOREHOLE/TEST PIT LOCATION PLAN

DATE
NOVEMBER 2023

SCALE:
NTS

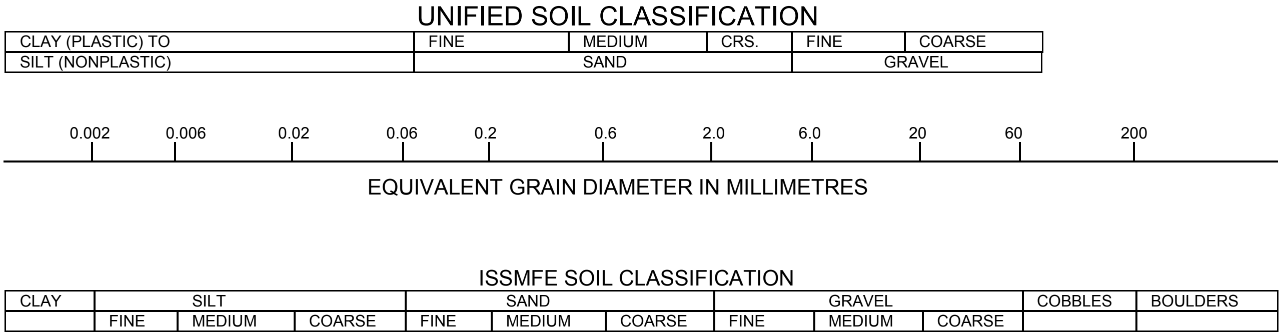
DWG NO.
A-2

Nov 07, 2023 - 9:19am \\exp\data\SUD\SUD-23012250-A0\60 Execution\65 Drawings\SUD-23012250-A0 - BOREHOLE LOCATION PLAN.dwg

Appendix B – Borehole and Test Pit Logs

Notes on Sample Descriptions

1. All sample descriptions included in this report follow the International Society for Soil Mechanics and Foundation Engineering (ISSMFE), as outlined in the Canadian Foundation Engineering Manual. Note, however, that behavioral properties (i.e. plasticity, permeability) take precedence over particle gradation when classifying soil. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually. Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems.



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

3. Till: The term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (75 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Notes On Soil Descriptions

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay and Silt	<0.060 mm	“trace” (e.g. Trace sand)	1% to 10%
Sand	0.060 to 2.0 mm	“some” (e.g. Some sand)	10% to 20%
Gravel	2.0 to 75 mm	adjective (e.g. sandy, silty)	20% to 35%
Cobbles	75 to 200 mm	“and” (e.g. and sand)	35% to 50%
Boulders	>200 mm		

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

Cohesionless Soil		Cohesive Soil		
Compactness	**Standard Penetration Resistance “N” Blows / 0.3 m**	**Consistency**	**Undrained Shear Strength (kPa)**	**Standard Penetration Resistance “N” Blows / 0.3 m**
Very Loose	0 to 4	Very soft	<12	<2
Loose	4 to 10	Soft	12 to 25	2 to 4
Compact	10 to 30	Firm	25 to 50	4 to 8
Dense	30 to 50	Stiff	50 to 100	8 to 15
Very Dense	Over 50	Very Stiff	100 to 200	15 to 30
		Hard	>200	>30

5. **ROCK CORING**

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

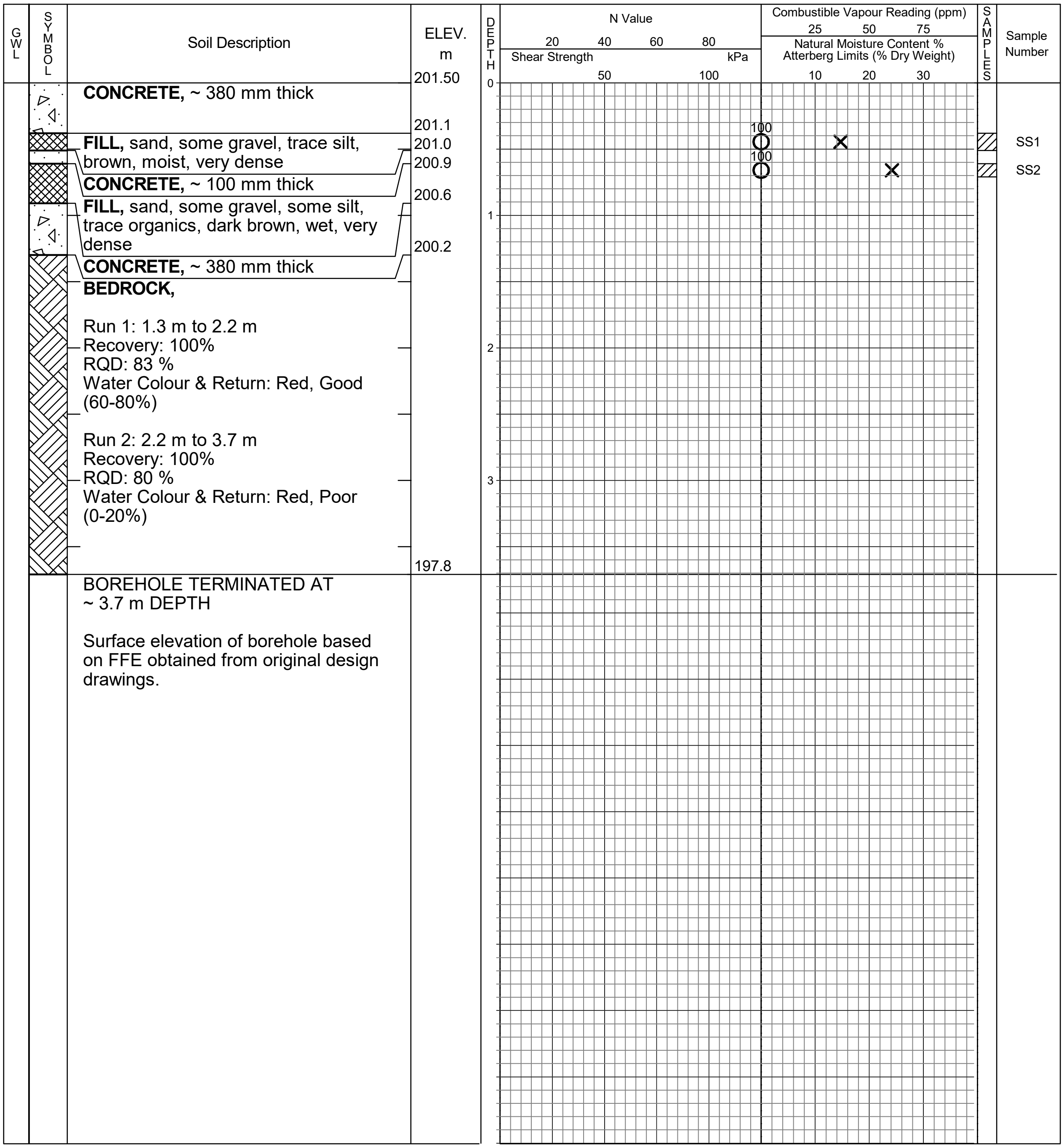
RQD Classification	RQD (%)
Very Poor Quality	<25
Poor Quality	25 to 50
Fair Quality	50 to 75
Good Quality	75 to 90
Excellent Quality	90 to 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

Log of Borehole BH-1

Project No. SUD-23012250-A0
Figure No. B-2
Project: ONTC Paint Shop and Exterior Upgrades
Sheet No. 1 of 1
Location: 915 McIntyre Street, North Bay, Ontario
Interior Borehole
Date Drilled: October 19, 2023
Drill Type: Dietrich D25
Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	―
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	✕
Plastic and Liquid Limit	⊢○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲



Soil Description	ELEV. m	Sample Number
	201.50	
CONCRETE, ~ 380 mm thick	201.1	
FILL, sand, some gravel, trace silt, brown, moist, very dense	201.0 200.9	SS1
CONCRETE, ~ 100 mm thick		SS2
FILL, sand, some gravel, some silt, trace organics, dark brown, wet, very dense	200.6	
CONCRETE, ~ 380 mm thick	200.2	
BEDROCK, Run 1: 1.3 m to 2.2 m Recovery: 100% RQD: 83 % Water Colour & Return: Red, Good (60-80%) Run 2: 2.2 m to 3.7 m Recovery: 100% RQD: 80 % Water Colour & Return: Red, Poor (0-20%)	197.8	

BOREHOLE TERMINATED AT ~ 3.7 m DEPTH

Surface elevation of borehole based on FFE obtained from original design drawings.

SUDBURY GEO SUD-23012250-A0 ONTC PAINT SHOP.GPJ NEW.GDT 11/10/23

EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

Time	Water Level (m)	Depth to Cave (m)
Upon Completion	N/A	N/A

Log of Borehole BH-2

Project No. SUD-23012250-A0

Figure No. B-3

Project: ONTC Paint Shop and Exterior Upgrades

Sheet No. 1 of 1

Location: 915 McIntyre Street, North Bay, Ontario

Interior Borehole

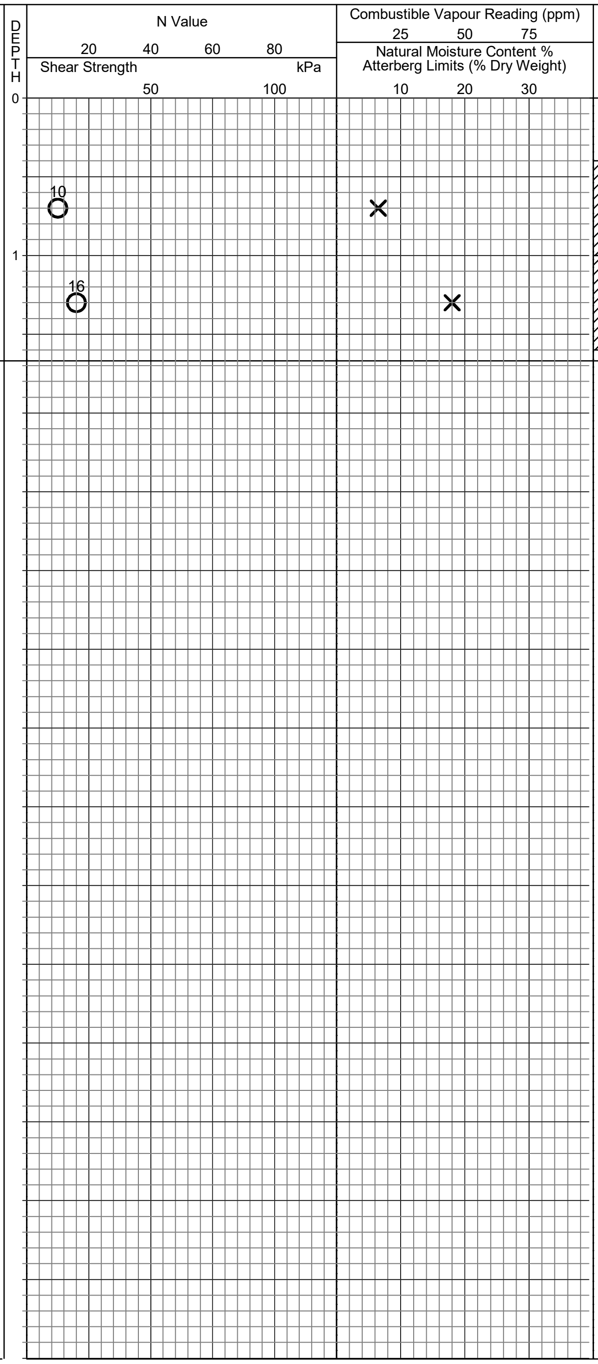
Date Drilled: October 19, 2023

Drill Type: Dietrich D25

Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	✕
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	SAMPLES	Sample Number
		CONCRETE, ~ 410 mm thick	201.50	0		
		FILL, sand, trace gravel, trace silt, brown, moist, compact	201.1		▨	SS1
		wet below ~ 0.9 m depth		1	▨	SS2
		BOREHOLE TERMINATED AT ~ 1.7 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK Surface elevation of borehole based on FFE obtained from original design drawings.	199.8			



EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
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Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

Time	Water Level (m)	Depth to Cave (m)
Upon Completion	Dry	No Cave

SUDBURY GEO SUD-23012250-A0 ONTC PAINT SHOP.GPJ NEW.GDT 11/10/23

Log of Borehole BH-3

Project No. SUD-23012250-A0

Figure No. B-4

Project: ONTC Paint Shop and Exterior Upgrades

Sheet No. 1 of 1

Location: 915 McIntyre Street, North Bay, Ontario

Interior Borehole

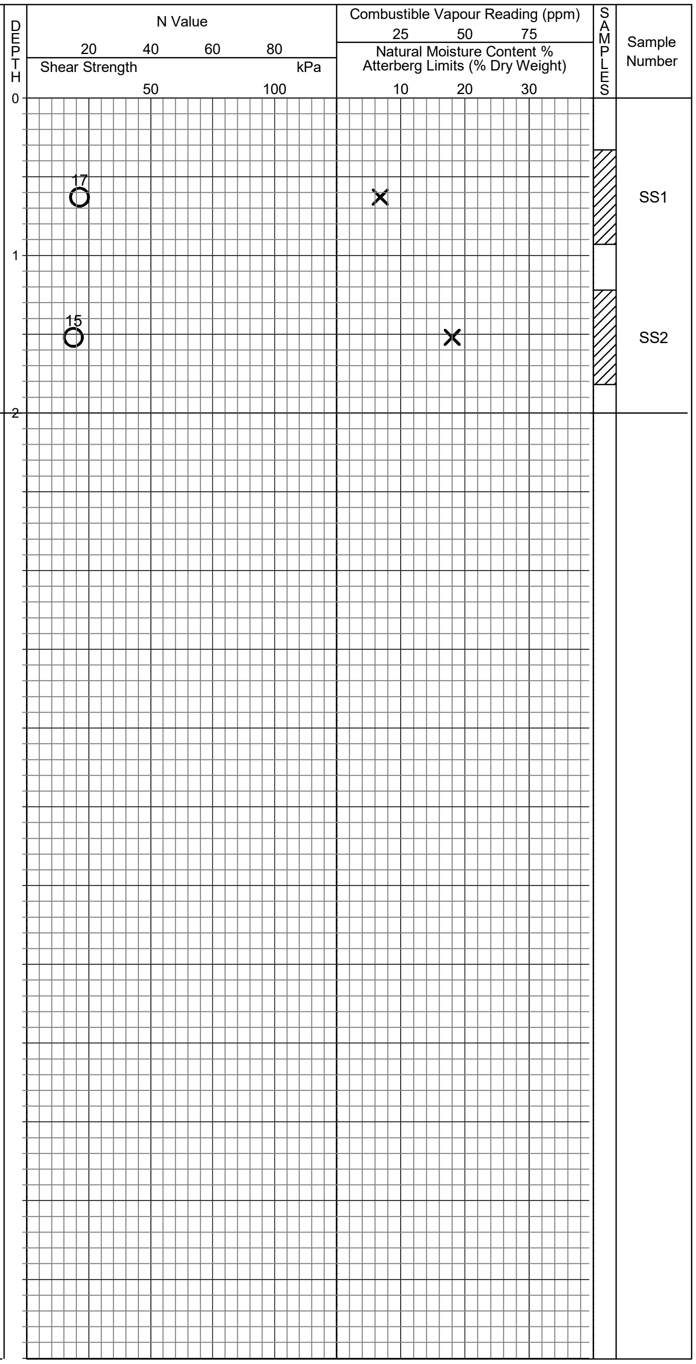
Date Drilled: October 19, 2023

Drill Type: Dietrich D25

Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	✕
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
			201.50
		CONCRETE, ~ 330 mm thick	201.2
	▨	**FILL,** sand, trace gravel, trace silt, brown, moist, compact brown to grey, moist to wet below ~ 1.2 m depth.	199.5
		BOREHOLE TERMINATED AT ~ 2.0 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK Surface elevation of borehole based on FFE obtained from original design drawings.	



EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

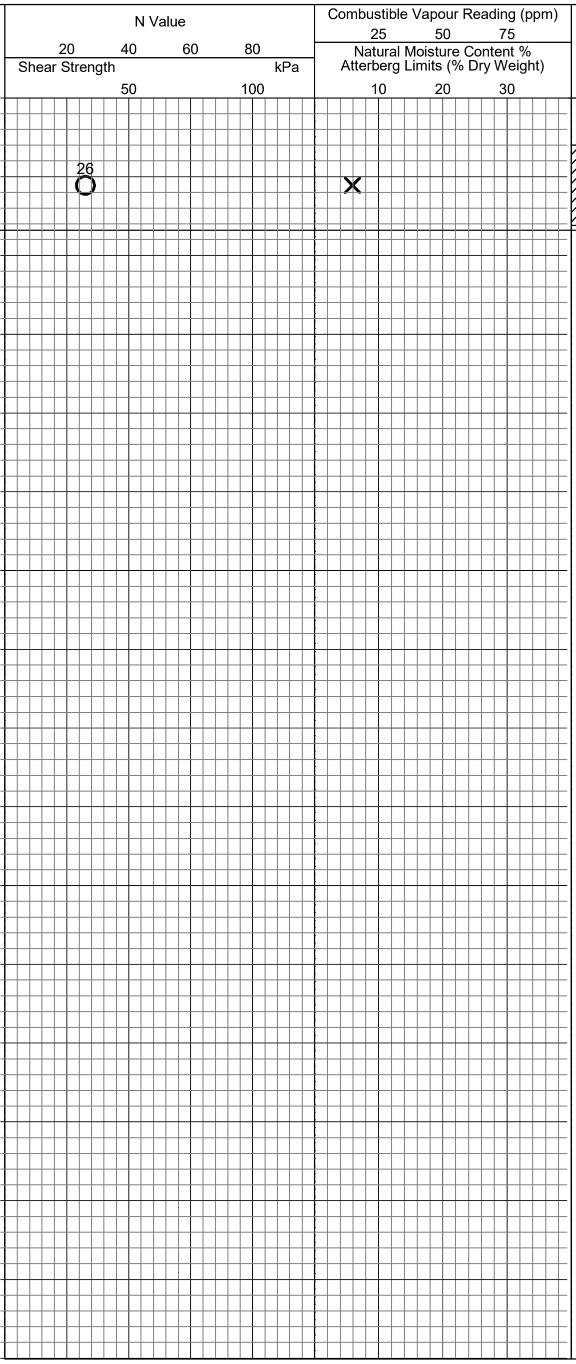
Time	Water Level (m)	Depth to Cave (m)
Upon Completion	Dry	No Cave

Log of Borehole BH-4

Project No. SUD-23012250-A0
Figure No. B-5
Project: ONTC Paint Shop and Exterior Upgrades
Sheet No. 1 of 1
Location: 915 McIntyre Street, North Bay, Ontario
Interior Borehole
Date Drilled: October 18, 2023
Drill Type: Dietrich D25
Datum: Geodetic (hand-held GPS)

Auger Sample	☒	Combustible Vapour Reading	☐
SPT (N) Value	○ ▨	Natural Moisture	✕
Dynamic Cone Test	—	Plastic and Liquid Limit	├─○
Shelby Tube	■	Undrained Triaxial at % Strain at Failure	⊕
Field Vane Test	✚ S	Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	SAMPLES	Sample Number
		CONCRETE, ~ 280 mm thick	201.50	0		
		FILL, sand, some gravel, trace silt, brown, moist, compact	201.2 200.7		▨	SS1
		BOREHOLE TERMINATED AT ~ 0.8 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK Surface elevation of borehole based on FFE obtained from original design drawings.				



EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

Time	Water Level (m)	Depth to Cave (m)
Upon Completion	Dry	No Cave

Log of Borehole BH-5

Project No. SUD-23012250-A0

Figure No. B-6

Project: ONTC Paint Shop and Exterior Upgrades

Sheet No. 1 of 1

Location: 915 McIntyre Street, North Bay, Ontario

Interior Borehole

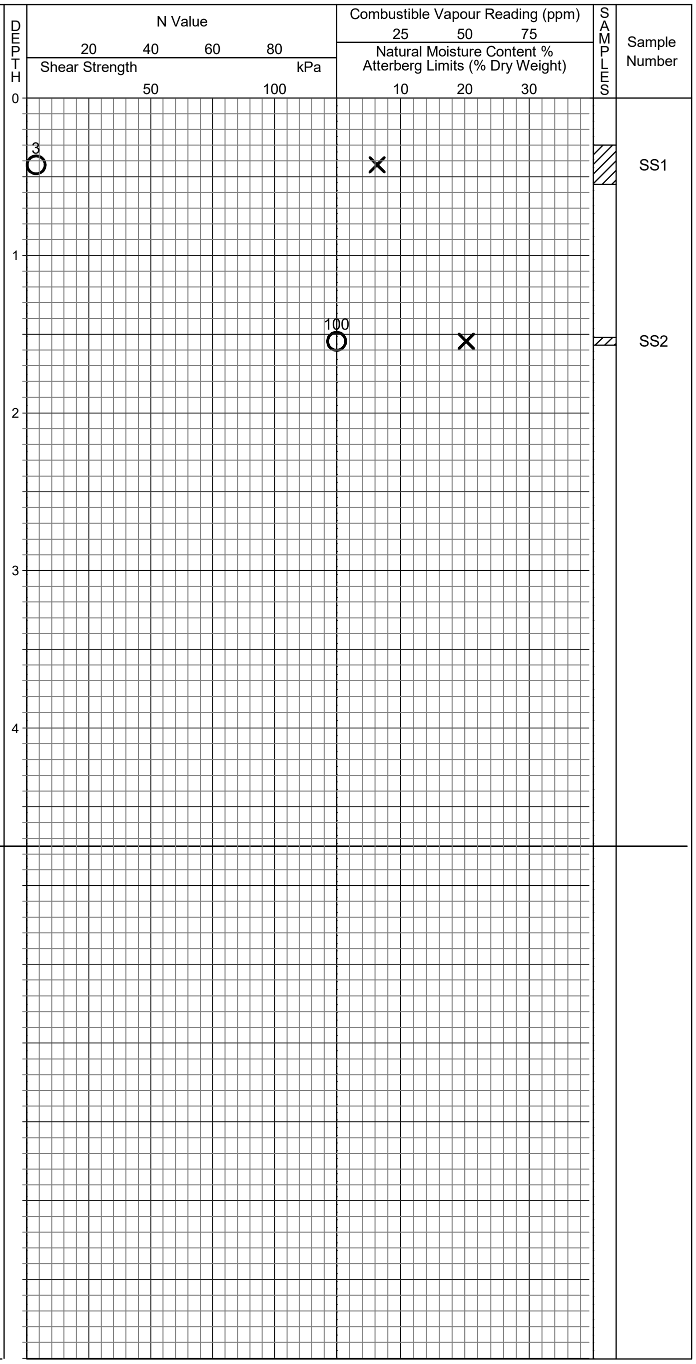
Date Drilled: October 18, 2023

Drill Type: Dietrich D25

Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

Soil Description	ELEV. m
	201.50
CONCRETE, ~ 305 mm thick	201.2
FILL, sand, trace gravel, trace silt, brown, moist, very loose	
brown to black, wet below ~ 1.5 m depth.	199.8
BEDROCK, Run 1: 1.7 m to 2.2 m Recovery: 100% RQD: 0% Water Colour & Return: Red, Poor (20-40%) Run 2: 2.2 m to 3.7 m Recovery: 100% RQD: 68% Water Colour & Return: Red, Good (40-60%) Run 3: 3.7 m to 4.8 m Recovery: 100% RQD: 71% Water Colour & Return: Red, Good (40-60%)	196.8
BOREHOLE TERMINATED AT ~ 4.8 m DEPTH Surface elevation of borehole based on FFE obtained from original design drawings.	



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CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

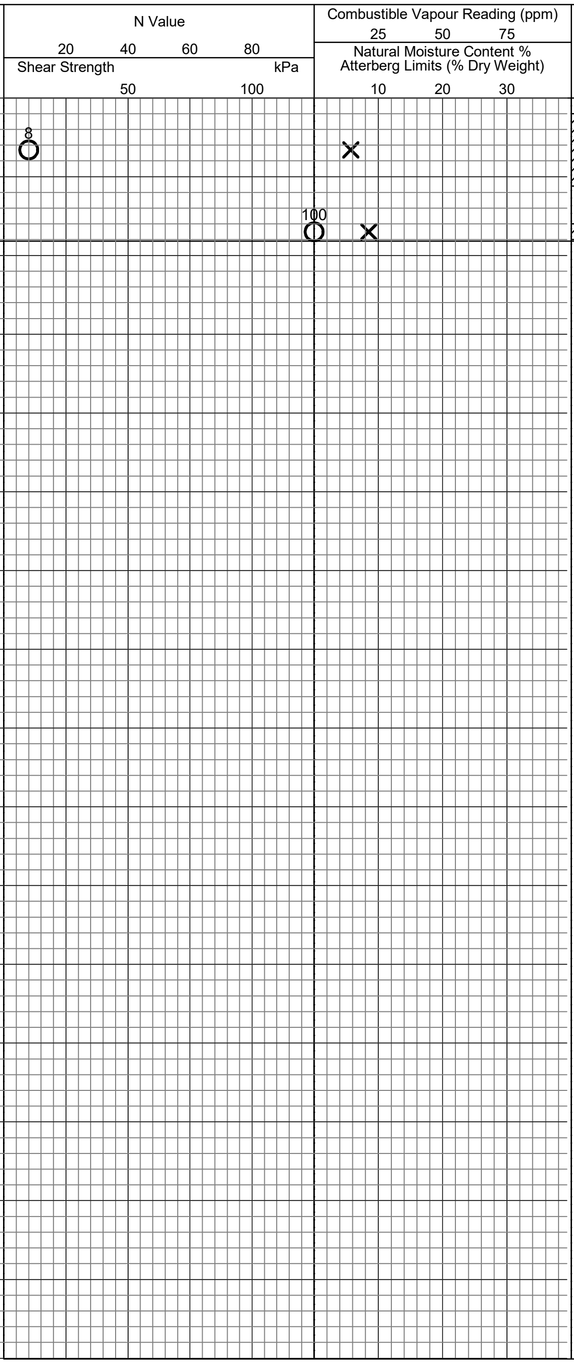
Time	Water Level (m)	Depth to Cave (m)
Upon Completion	N/A	N/A

Log of Borehole BH-6

Project No. SUD-23012250-A0
Figure No. B-7
Project: ONTC Paint Shop and Exterior Upgrades
Sheet No. 1 of 1
Location: 915 McIntyre Street, North Bay, Ontario
N5128469, E619172
Date Drilled: October 19, 2023
Drill Type: Dietrich D25
Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	□
Natural Moisture	✕
Plastic and Liquid Limit	├─○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	SAMPLES	Sample Number
			205.00	0		
	■	**ASPHALT,** ~ 100 mm thick	204.9			
	☒	**FILL,** sand and gravel, trace silt, brown to black, moist, loose	204.1		▨	SS1 SS2
		BOREHOLE TERMINATED AT ~ 0.9 m DEPTH DUE TO AUGER REFUSAL ON SUSPECTED BEDROCK				



EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

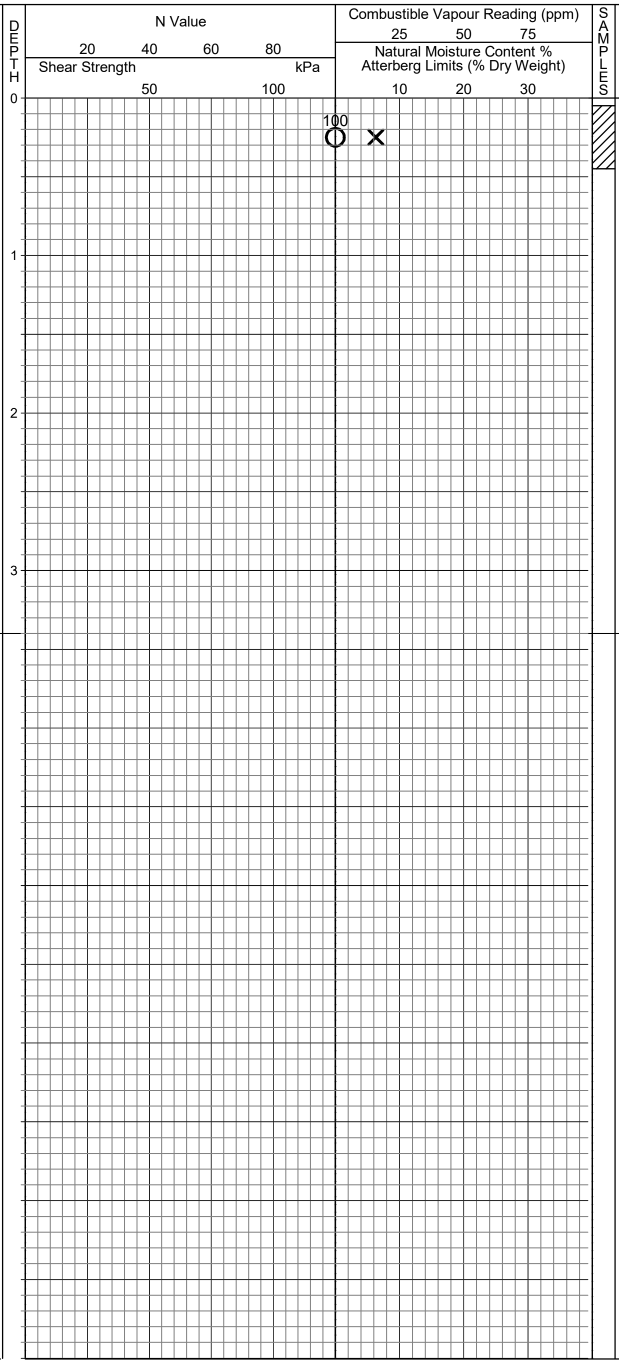
Time	Water Level (m)	Depth to Cave (m)
Upon Completion	Dry	No Cave

Log of Borehole BH-7

Project No. SUD-23012250-A0
Figure No. B-8
Project: ONTC Paint Shop and Exterior Upgrades
Sheet No. 1 of 1
Location: 915 McIntyre Street, North Bay, Ontario
N5128496, E619182
Date Drilled: October 19, 2023
Drill Type: Dietrich D25
Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	✕
Plastic and Liquid Limit	⊢—○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m	Sample Number
			206.00	
		ASPHALT, ~ 50 mm thick	206.0	
		FILL, sand, some gravel, trace silt, brown, moist		SS1
			205.4	
		BEDROCK, Run 1: 0.6 m to 1.9 m Recovery: 100% RQD: 84% Water Colour & Return: Red to Grey, Good (40-60%) Run 2: 1.9 m to 3.4 m Recovery: 100% RQD: 73% Water Colour & Return: Grey, Good (50-70%)		
			202.6	
		BOREHOLE TERMINATED AT ~ 3.4 m DEPTH		



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885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

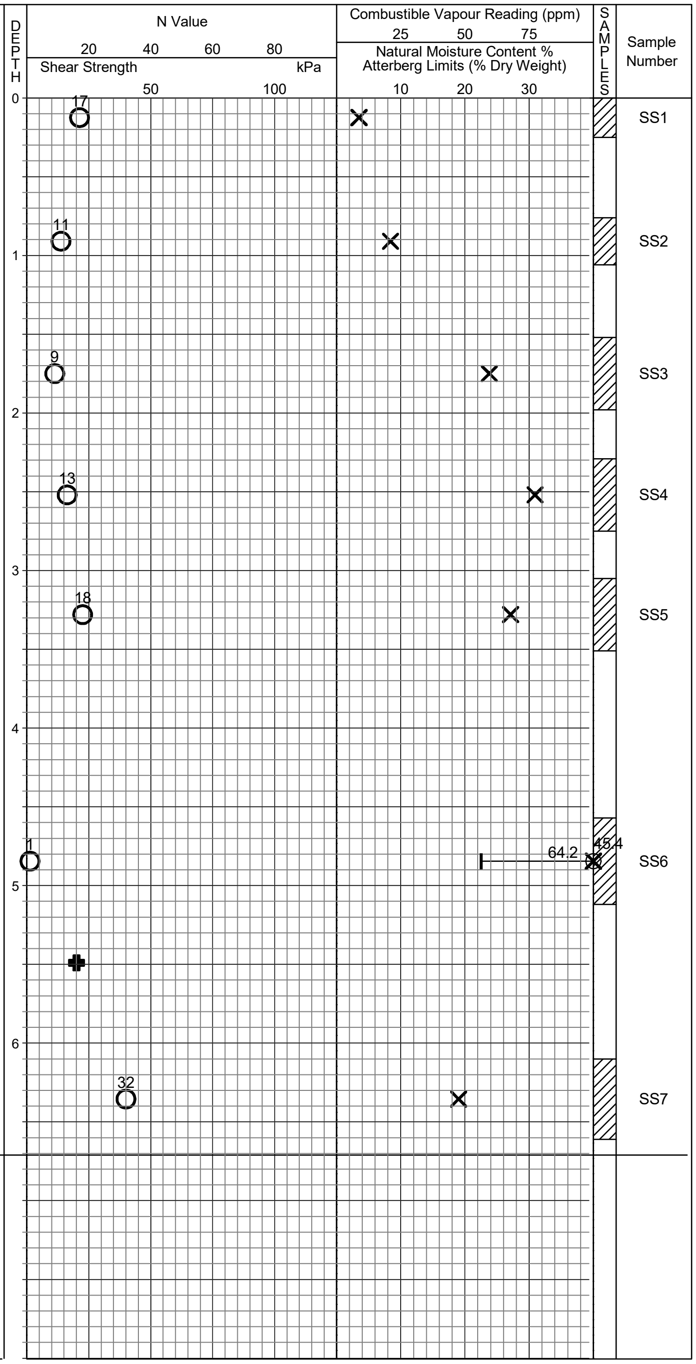
Time	Water Level (m)	Depth to Cave (m)
Upon Completion	N/A	N/A

Log of Borehole BH-8

Project No. SUD-23012250-A0
Figure No. B-9
Project: ONTC Paint Shop and Exterior Upgrades
Sheet No. 1 of 1
Location: 915 McIntyre Street, North Bay, Ontario
N5128355, E619169
Date Drilled: October 19, 2023
Drill Type: Dietrich D25
Datum: Geodetic (hand-held GPS)

Auger Sample	☒
SPT (N) Value	○ ▨
Dynamic Cone Test	—
Shelby Tube	■
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	✕
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕
Penetrometer	▲

GWL	SYMBOL	Soil Description	ELEV. m
		FILL, sand, some gravel, some silt, brown and black, moist, compact	202.00
		SILTY SAND, trace gravel, brown to grey, wet, loose	200.5
		SAND, some silt, grey, wet, compact	199.7
		CLAY AND SILT, trace sand, grey, wet, medium plasticity, very soft to soft	197.4
		TILL, silty sand, some gravel, grey, wet, dense	195.9
		BOREHOLE TERMINATED AT ~ 6.7 m DEPTH	195.3



SUDBURY GEO SUD-23012250-A0 ONTC PAINT SHOP.GPJ NEW.GDT 11/10/23

exp.
EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Borehole data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

Time	Water Level (m)	Depth to Cave (m)
Upon Completion	Dry	No Cave

Log of Test Pit TP-1

Project No. SUD-23012250-A0

Figure No. B-10

Project: ONTC Paint Shop and Exterior Upgrades

Sheet No. 1 of 1

Location: 915 McIntyre Street, North Bay, Ontario
N5128508, E619189

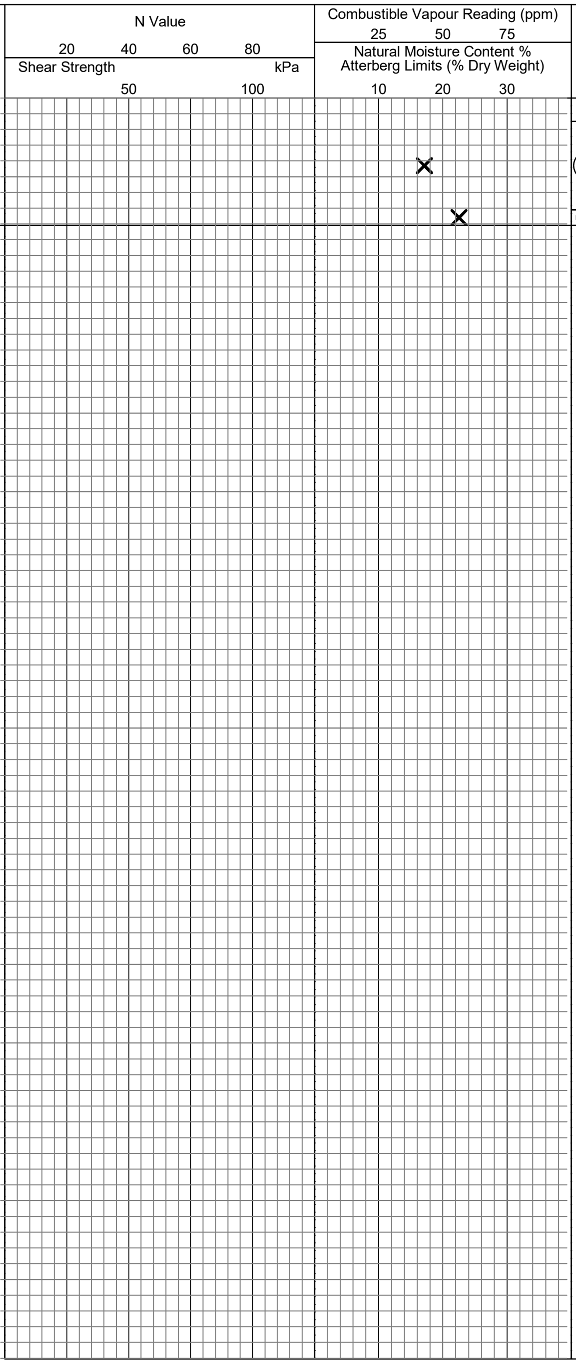
Date Excavated: October 18, 2023

Excavator Type: Excavator

Datum: Geodetic (hand-held GPS)

Grab Sample	G
Penetrometer	▲
Field Vane Test	✚ S
Combustible Vapour Reading	☐
Natural Moisture	×
Plastic and Liquid Limit	├──○
Undrained Triaxial at % Strain at Failure	⊕

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	SAMPLES	Sample Number
	▨		206.00	0		
	▨	**FILL,** sand, some silt, trace gravel, brown, moist	205.9			
	▨	**FILL,** silty sand, trace to some gravel, trace clay, black (stained), moist			G	SA1
▼	▨	brown, no clay below ~ 0.7 m depth.	205.2 205.20		G	SA2
		TEST PIT TERMINATED AT ~ 0.8 m DEPTH DUE TO REFUSAL ON BEDROCK Water trickling in at base of test pit				



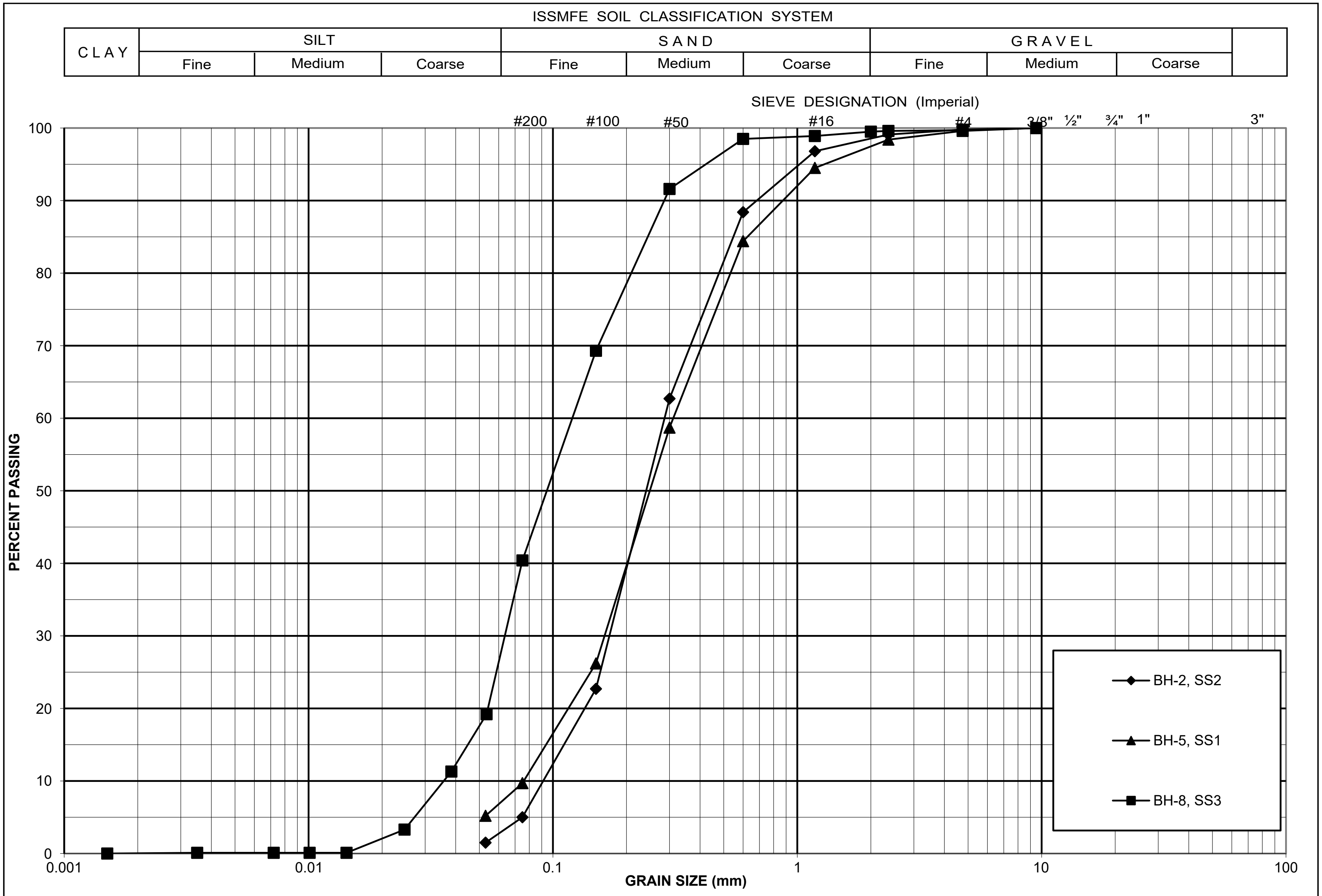
EXP Services Inc.
885 Regent Street
Sudbury, ON P3E 5M4
CANADA
t: +1.705.674.9681
f: +1.705.674.5583

Test Pit data requires interpretation assistance from EXP before use by others.

See Figures B-1A and B-1B for Notes on Sample Description

Time	Water Level (m)	Depth to Cave (m)
Upon Completion	0.8	No Cave

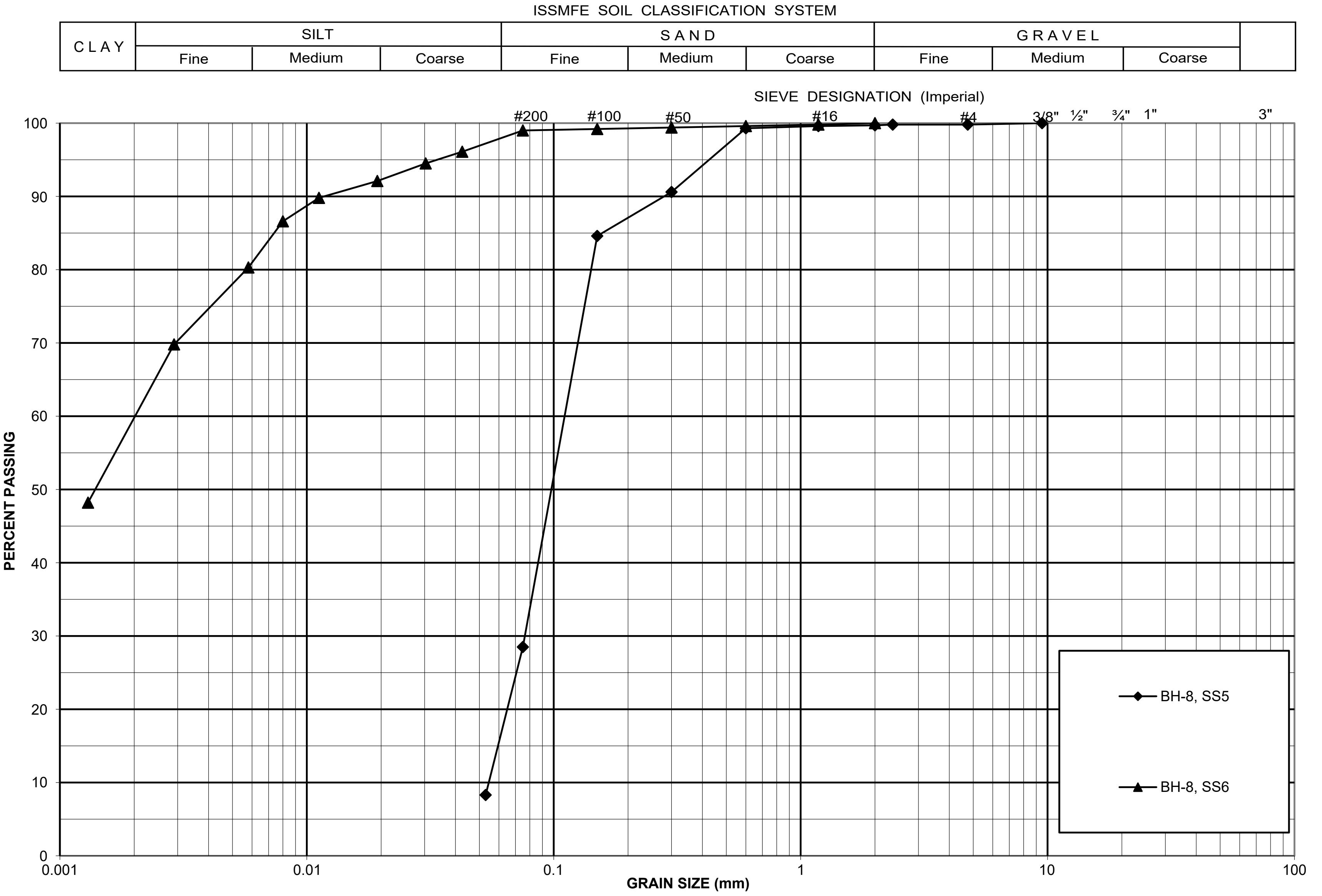
Appendix C – Laboratory Test Results



GRAIN SIZE DISTRIBUTION

Paint Shop and Exterior Upgrades
915 McIntyre Street, North Bay, Ontario

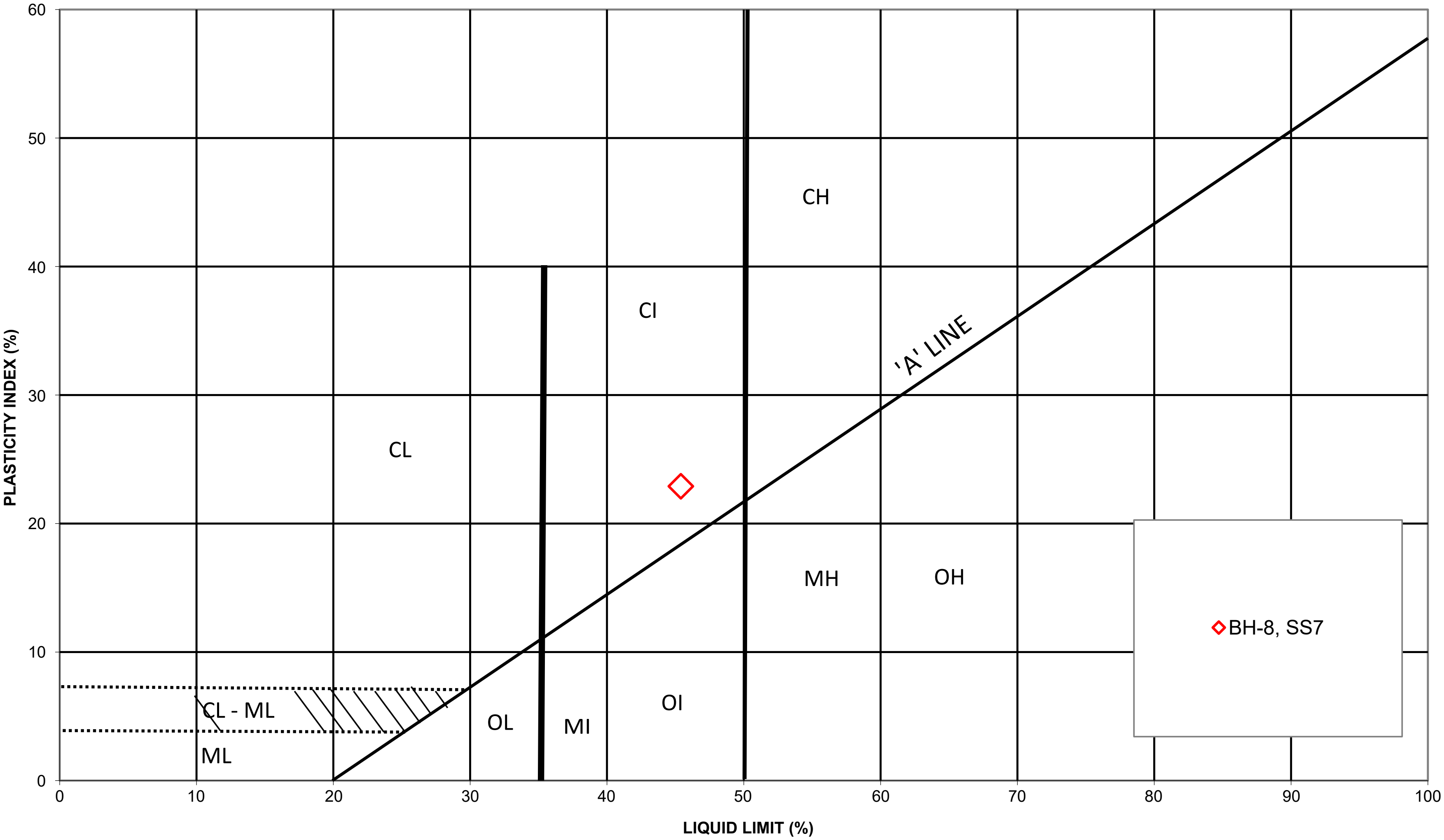
FIGURE: C-1
PROJECT No: SUD-23012250-A0
DATE: November 2023



GRAIN SIZE DISTRIBUTION

Paint Shop and Exterior Upgrades
915 McIntyre Street, North Bay, Ontario

FIGURE: C-2
PROJECT No: SUD-23012250-A0
DATE: November 2023



PLASTICITY CHART

Paint Shop and Exterior Upgrades
915 McIntyre Street, North Bay, Ontario

FIGURE: C-3

PROJECT No: SUD-23012250-A0

DATE: November 2023

exp Services Inc.
885 Regent Street
Sudbury, Ontario
P3E 5M4
Telephone: (705) 674-9681
Facsimile: (705) 674-8271

SUMMARY OF ROCK CORE TEST DATA

ASTM D7012 - 14 (Method C)

CLIENT: Ontario Northland Transportation Commision

JOB NUMBER: SUD-23012250-A0

JOB NAME: ONTC Paint Shop

DATE: November 7, 2023

LAB No.	22328	22329
CORE LOCATION	BH 1-Run 1	BH 5-Run 2
DEPTH	6'3 1/2"-6' 8"	9'6"-9'10 1/2"
DATE TESTED	7-Nov-23	7-Nov-23
LENGTH (mm)	115.5	117.5
DIAMETER (mm)	47.5	47.5
DENSITY (kg/m^3)	**2692**	**2886**
COMPRESSIVE STRENGTH (MPa)	153.6	128.3
TYPE OF FRACTURE	**CONE**	**SHEAR**
CONDITION AT TIME OF TESTING	**DRY**	**DRY**

COMMENTS:

DISTRIBUTION: