



## Geotechnical Investigation and Design Report

*Municipality of Callander*

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Proposed New Operations Building  
Callander, Ontario

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## 1. Introduction

Further to our Proposal No. 26-024-GP, dated February 12, 2026, and your subsequent authorization to proceed, EXP Services Inc. (EXP) has completed the field investigation and the geotechnical engineering evaluation for the proposed buildings. Our comments and recommendations, based on the results of the field investigation and our understanding of the project scope are provided in this report.

It is understood by EXP the two new buildings are to be constructed at the Callander Operations Yard. One building is proposed to be a salt storage structure located to the northeast of the site, in the area of the existing garage (to be demolished), and the other is proposed to be a garage located on the southeast area of the site. Boreholes were completed at locations noted on the attached drawing, Dwg. No. A-1, included in Appendix A.

## 2. Field Investigation

The field investigation for this project consisted of the advancement of five (5) sampled boreholes within the proposed development site. The boreholes were advanced on February 23 and 24, 2025, with the borehole logs found in Appendix B, Figs. B-2 to B-6. The advancement of the boreholes was supervised on a full-time basis by a geotechnical representative from EXP.

The sampled boreholes were advanced using a truck mounted, CME 75 drill rig equipped with a hollow stem auger and split spoon sampling equipment, in locations free of buried or overhead services. Soils samples were then obtained directly from the augers within the first 0.75 m intervals thereafter in conjunction with the Standard Penetration Test (SPT), at depths noted on the attached borehole logs in Appendix B. The SPT "N" values have been recorded at each sample interval to provide an assessment of the in-situ compactness condition of the subgrade soils. A piezometer was installed in borehole BH-5.

Groundwater levels were measured within the open boreholes prior to backfilling and on the day after installation in the piezometer. Boreholes were backfilled with bentonite chips and auger cuttings.

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 locations were obtained by handheld GPS during the field investigation. The elevations of the boreholes surveyed to a temporary benchmark, the finished floor of existing building on the northwest portion of the site. The borehole locations and elevations should be considered accurate only to the degree implied by the methods used and should not be used for design purposes.

## 3. Laboratory Testing

A laboratory testing program was performed on representative soil samples and consisted of moisture content determinations. The laboratory test results are summarized on the attached borehole 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 borehole 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 Proposed Salt Storage Structure Soil Conditions

Boreholes BH-1 and BH-2 were drilled within the footprint of the proposed salt storage structure.

Borehole BH-1 encountered a 50 mm thick layer of asphalt at surface.

Underlying the asphalt in borehole BH-1 and from surface in borehole BH-2, a layer of fill was encountered. This fill varied from sand some gravel, trace silt to sand some silt, was brown in colour and frozen. This material was frozen, and the uncorrected SPT "N" values within the material were greater than 100 blows per 300 mm, and not representative of the compactness condition. The moisture content of the ranged between 1 to 22%.

Refusal was encountered in boreholes BH-1 and BH-2 at 1.2 and 1.1 m respectively on suspected bedrock or boulders. The boreholes BH-1 and BH-2 were dry with no cave.

### 4.2 Proposed Garage Soil Conditions

Boreholes BH-3 to BH-5 were completed within the footprint of the proposed garage.

Boreholes BH-3 to BH-5 encountered fill at surface. The fill varied from a sand some silt to a sand, some gravel, some silt, to sand and gravel. Wood debris and organics were noted in the fill. The fill extended to between 2.3 to 3.0 m. The fill was brown, frozen and moist to wet. Uncorrected SPT "N" values within the fill varied between 6 and greater than 100 blows per 300 mm, classifying the soil as loose to very dense in compactness conditions. The moisture content of the fill ranged between 4 to 95%.

Underlying the fill at, boreholes BH-3 to BH-5, a cohesionless silt was encountered. The silt, contained some sand and trace clay. The cohesionless soils were brown, moist to wet. This layer extended to the termination depth of the boreholes. Uncorrected SPT "N" values within the cohesionless soils varied between 6 and greater than 100 blows per 300 mm, classifying the soil as loose to very dense in compactness conditions. The moisture content of the cohesionless soils ranged between 13 to 41%.

Refusal on suspected bedrock or boulders was encountered in borehole BH-5 at a depth of 4.3 m.

The groundwater level in the boreholes upon completion ranged between 2.3 to 2.6 m. In the piezometer installed piezometer, the ground water level on February 24, 2026 and March 3, 2026, the water level was 2.6 m below 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. Foundation Recommendations

Based on the soil conditions encountered within the boreholes, it is recommended that the proposed buildings be founded conventional strip or spread footings bearing on the encountered native soils or bedrock or engineered fill over native soils or bedrock.

Note that the proposed foundation details and loading conditions have not been provided to EXP at the time of this report. EXP should be retained to review the final designs and specifications to confirm that they are in general agreement with the assumptions on which our recommendations are based. If not accorded the privilege of making this review, EXP will assume no responsibility for interpretation of the recommendations in this report.

### 5.1 Conventional Strip or Spread Footings on Native Soils (Proposed Garage Building)

Prior to the placement of the footings, topsoil, organics, fill and any other deleterious material must be removed down to the undisturbed native silt. The exposed subgrade should be proof rolled to identify any soft or unstable areas. The exposed subgrade and proof rolling is to be inspected by a representative from EXP prior to placing fill material or concrete. Any soft or loose areas encountered below the footing 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 the Granular "A" or Granular "B" Type II to stabilize the native soils.

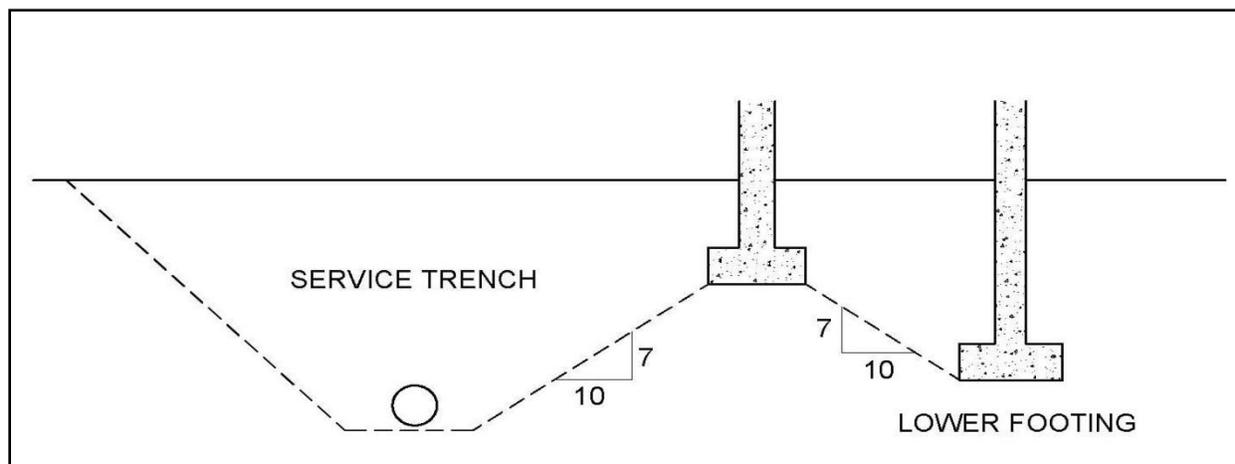
To protect the footing base from construction activity or inclement weather, a 150 mm thick layer of Granular "A" material (OPSS 1010) can be placed directly below the footings and extend a minimum of 300 mm on either side of the footing edge and then slope down at 1H:1V. In-lieu of the Granular "A", a lean mix concrete base can be poured. The lean mix concrete should extend a minimum of 300 mm on either side of the footings. Note that the footing base should not be left exposed beyond the day of excavation.

Engineered fill can be placed between the native soils and footings to raise the base of footing elevation if necessary. The engineered fill is to consist of Granular "B" Type II (OPSS 1010). A final 150 mm thick layer of Granular "A" (OPSS) should be placed directly below the footing. The engineered fill below any footings is to extend horizontally a minimum of 300 mm from any footing edge and then slope down at 1H:1V to the underlying native soils.

All engineered fill is to be placed in maximum 150 mm thick lifts and is to be compacted to 100% of the Standard Proctor Maximum Dry Density (SPMDD) within 1.5% of optimum moisture content.

Footings founded on the undisturbed native soils, or on engineered fill overlying undisturbed native soils, can be designed with a factored geotechnical resistance at Ultimate Limit States (ULS) of 150 kPa. This value was calculated using a geotechnical resistance factor of 0.5. A bearing pressure at Serviceability Limit States (SLS) of 100 kPa may be used. Footings designed with the recommendations contained herein are expected to settle less than 25 mm total and 20 mm differential.

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 horizontals to 7 vertical from the near edge of a lower foundation or bottom of a service trench, as indicated on Figure 5-1 below.



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

These foundation recommendations assume the structures are lightly loaded. Strip and spread footing widths must comply with minimum Code requirements.

## 5.2 Footings on Engineered Fill Overlying Bedrock (Salt Storage Structure)

Pending final site grades, foundations on engineered fill overlying bedrock can be considered and may be designed for a factored geotechnical resistance at Ultimate Limit States (ULS) of 300 kPa and a geotechnical reaction at Serviceability Limit States (SLS) of 200 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 200 kPa, total settlements are significantly less than the typically acceptable level of 25 mm total.

All existing fills would be required to be removed from the foundation area, including the slab area to expose sound 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 overlying bedrock is 300 mm.

The engineered fill pad, is to extend laterally a minimum of 1.0 m beyond the edge of the foundation and 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 a qualified geotechnical engineer to ensure that the recommendations contained herein are met.

All bedrock surfaces must be reviewed by a qualified geotechnical engineer 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.

### 5.3 Footings on Bedrock (Salt Storage Structure)

Conventional strip or spread foundations bearing on sound bedrock, can be designed with a factored geotechnical resistance at Ultimate Limit States (ULS) of 1 MPa, calculated using a geotechnical resistance factor of 0.5. A bearing pressure at 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 a qualified geotechnical engineer prior to pouring foundation concrete. This is necessary to verify the assumed foundation bearing conditions and review the foundation construction procedures, bedrock slope, etc.

#### Rock Dowels and Anchors

If dowels are required, the structural engineer normally designs the length and diameter of the steel dowels for footings, based on the type of bedrock and its strength parameters.

For bedrock in the Sudbury 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. The bond length or grouted portion of the dowel for this rock mass should be a minimum of 3.0 m. 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, " $\tau_b$ " 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).

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:

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

where

$$\begin{aligned} P &= \text{working capacity of anchor (kN)} \\ \tau_b &= \text{working bond} \\ d &= \text{core hole diameter (m)} \end{aligned}$$

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. Weathered bedrock was not encountered in the explored boreholes, however surficial weathered rock has previously been encountered in the area and therefore the above design assumption is recommended.

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 a qualified testing agency.

## 5.4 Lateral Earth Pressure

Any foundations and any 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:

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

where,

$$\begin{aligned} p &= \text{Lateral earth pressure (kPa)} \\ K &= \text{Coefficient of earth pressure} \\ \gamma &= \text{Unit weight of backfill (kN/m}^3\text{)} \\ \gamma_w &= \text{Unit weight of water (kN/m}^3\text{)} \\ h &= \text{Depth to point of interest (m)} \\ h_w &= \text{Depth of water above point of interest (m)} \\ q &= \text{Surcharge load acting adjacent to the wall at the ground surface (kPa)} \end{aligned}$$

Table 5-1 lists various earth pressure properties for given materials.

**Table 5-1:** Material Types and Earth Pressure Parameters

Material	Friction Angle $\phi'$ (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 (kN/M <sup>3</sup> )
Granular “A”	38°	0.24	4.2	0.38	22
Granular “B”	35°	0.27	3.7	0.43	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 considered 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.

## 5.5 Frost Considerations

The freezing index in the Callander area is approximately 1220 C degree-days. There is potential for up to 2.1 m of frost penetration to occur over the winter months in unprotected, unheated areas and 1.7 m for heated structures.

As such, foundations for unheated structures should be provided with a minimum of 2.1 m of earth cover frost protection and heated structures should be provided with 1.7 m of earth cover frost protection. Note that to be considered a heated structure; the building must be maintained continuously at a minimum temperature of 18°C. If this will not occur, the building/structure shall be considered unheated.

Should sufficient earth cover not be provided, insulation will 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 HI 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.

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

## 5.6 Site Classification for Seismic Response

The Ontario Building Code (OBC) has adopted the National Building Code of Canada requirements for seismic design considerations. Based on the conditions encountered at the borehole locations, the Site Classification for Seismic Response has been estimated to be Site Class C for the proposed salt storage structure and Site Class D for the proposed garage as per the OBC clause 4.1.8.4, Site Properties and Table 4.1.8.4 A, Site Classification for Seismic Response.

These earthquake/seismic design parameters should be reviewed in detail by the structural engineer and incorporated into the design as required. If a Site Classification based on shear wave velocity testing is required, EXP can provide a quote to perform the necessary testing.

## 5.7 Floor Slab-on-Grade

Floor slab-on-grade construction will be possible at this site for both building locations, provided that that all topsoil, fill, existing footings, walls, services, organics, and other deleterious materials are removed down to competent native soils and/or bedrock. The subgrade soils should be proof-roll compacted in the presence of EXP prior to placing any engineered fill and exposed bedrock should be visually reviewed to ensure it has been properly cleaned. Any soft areas encountered during proof-rolling should be excavated and replaced with Granular "B" Type II (OPSS 1010) material.

Once the native ground surface and/or bedrock is prepared, all required up-fill material is to consist of Granular "B" Type I or II (OPSS 1010). If wet soil conditions are present, a non-woven geotextile separator (Terrafix 270R or equivalent) should be placed between the subgrade soils and the Granular "B" material to stabilize the native soils. A final 300 mm thick layer of 19 mm minus clearstone (OPSS 1004) or Granular "A" (OPSS 1010) should be placed directly below the floor slab-on-grade combined with an appropriate moisture barrier such as a polyethylene membrane. All fill material should be placed in maximum 150 mm thick lifts and be compacted to 100% of the SPMDD (Standard Proctor maximum dry density) within 1.5% of the optimum moisture content.

## 5.8 Surface Drainage

The exterior grade around the buildings should be sloped away from the walls to prevent surface runoff from entering the building. Permanent perimeter weeping tile should be installed where any floor is less than 150 mm above final grade and is required to be dry. The drainage tile should have a minimum diameter of 100 mm, and be surrounded by well-draining filter material (i.e. 20 mm Clear Stone gravel). The filter material should be surrounded with a non-woven geotextile. The perforated drainage tile should drain to a suitable drainage area or interior sump. All subsurface walls should be adequately damp-proofed above the water table and waterproofed below the water table. The roof drains should discharge away from the building to appropriate drainage areas.

## 5.9 Pavement Structure Design Recommendations

The recommended pavement structure designs for both light traffic and heavy traffic areas are provided below. The recommended pavement structures outlined below assume adequate provision for drainage. A conventional asphalt pavement structure as noted below will typically have a functional service life of 18 years. This represents the number of years to the first rehabilitation (via overlay or resurfacing), assuming that regular maintenance and crack sealing is completed. Subsequent resurfacing is typically expected to last at least 10 years.

Layer	Light Traffic or Parking Areas	Heavy Traffic or Loading Areas
<b>Asphalt</b>	50 mm SP 12.5 Surface Course	40 mm SP 12.5 Surface Course <u>50 mm SP 19.0 Binder Course</u> 90 mm Total Thickness
<b>Base</b>	150 mm Granular "A"	150 mm Granular "A"
<b>Subbase</b>	300 mm Granular "B" Type II Or 450 mm Granular "B" Type I	450 mm Granular "B" Type II Or 600 mm Granular "B" Type I

Rigid pavement can be considered in areas of sharp truck turning or where heavy loads will be situated. The rigid pavement structure should include 200 mm of concrete over a 100 mm thick OGDL (Open Graded Drainage Layer) and a 200 mm thick base course, consisting of Granular "A" over the subbase material to improve the support and function as a drainage layer.

The roadway granular base and sub-base materials must be in accordance with OPSS 1010 and must be placed in maximum 150 mm lifts and compacted to 100% of the Standard Proctor Maximum Dry Density (SPMDD) at a moisture content within 2.0% of the optimum moisture content.

The long-term performance of pavement structures is highly dependent upon the sub-grade support conditions. Stringent construction control procedures should be maintained to ensure that uniform sub-grade moisture and density conditions are achieved. In addition, the need for adequate drainage cannot be overemphasized. The finished surface and underlying sub-grade must be sloped to provide effective drainage to catchbasins, ditching, and/or subdrains etc.

Surface water should not be allowed to pond along the outside edges of paved areas. Sub-drains should be installed to intercept excess subsurface moisture and prevent sub-grade softening.

Additional comments on the construction of the pavement structures are as follows:

- To ensure maximum service life of the pavement structures, all organics/peat and other deleterious materials should be removed to the native subgrade. An upfill required below the pavement structure can consist of Granular "B" Type I or II or a Select Subgrade Material (SSM) in accordance with OPSS 1010.
- Any subgrade soils should be proof-roll compacted prior to placing any engineered fill. Any soft areas encountered during proof-rolling should be excavated and replaced with a Granular "A" or Granular "B" Type II (OPSS 1010) material.
- If ditches are utilized, they should have inverts of at least 600 mm below the bottom of the sub-base.
- The most severe loading conditions on a soil pavement structure sub-grade usually occur during construction. Consequently, special provisions such as additional granular sub-base, may be required, especially if construction is completed during unfavorable weather conditions over native soils. Typically, the first lift of engineered fill is placed with a thickness of 300 mm prior to vibratory compaction to mitigate disturbance of the sub-grade soils.
- If wet soil conditions are present during construction, a non-woven geotextile separator (Terrafix 270R or equivalent) should be placed between the subgrade soils and any upfill/pavement structure material to stabilize the native soils.

## 6. Excavations

The in-situ native soils may be classified as Type 3 soils for excavations terminating above the groundwater level and Type 4 soils for excavations terminating below 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. Excavation side slopes in Type 4 soils should remain stable at a slope of 3H:1V.

Due to potential weathered bedrock, deeper excavations to sound bedrock must not be overlooked, including the requirement for temporary dewatering to remove any perched water that may be present between bedrock ridges.

For the encountered bedrock, the method selected for excavation will depend on the local block size and degree of weathering of the rock. In areas where weathering is not present, explosives may be required to break or to loosen the rock. Hoe Ramming may be used where minimal rock removal is required.

The need to excavate flatter side slopes if excessively wet or soft/loose materials, or concentrated seepage zones 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. Stockpiles should be kept a sufficient distance from any soil excavation so as not to surcharge the excavation side slopes.

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 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. If different subsurface conditions are encountered at the time of construction, we recommend that EXP be contacted immediately to evaluate the conditions encountered.

## 7. Dewatering

Groundwater is not anticipated at the proposed salt storage structure. At the proposed garage, the ground water table can be expected to be 2.6 m below ground surface. As there are up to 3 m of fill or other deleterious material that needs to be removed, dewatering will be required. Note that groundwater encountered at 2.6 m may be perched groundwater within the fill. Perched groundwater may be possible to control via sumps and pumps. However, contractors should complete test pits to fully understand the groundwater on site. Deeper groundwater may require more detailed systems.

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. 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.

As extensive dewatering will be required, it is recommended to have a hydrogeological study completed for the site to further understand dewatering volumes, environmental impacts on adjacent water bodies, and if a permit to take water will be required to complete the dewatering program. Should a hydrogeological study be required, please contact EXP to further

## 8. Re-Use of Excavated Material

The encountered soils are too poorly graded or fine grained to be re-used as free draining engineered fill. All in-situ materials may be used for general landscaping purposes away from structures/roads or in areas where non-free draining backfill may be required, provided it is environmentally safe to do so.

Any soils being removed from the site, must comply with the excess soil regulations (O.Reg. 406/19). While it is the responsibility of the source site to ensure soil exported off-site for reuse is suitable for the intended receiving site, it is highly recommended that the receiving site conduct an independent review of the analytical results to confirm the suitability of the soil to be reused at the specific receiving site.

## 9. Backfill

All imported backfill material used to backfill the foundation walls 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 used against the foundation walls should have no sizes greater than 75 mm and 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.

## 10. 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 is 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 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 because of freezing and thawing. The air entrainment will provide the capacity to absorb stresses during freeze/thaw action.

## 11. 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 all deleterious materials have been removed and to ensure that the actual conditions are not markedly different than 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.

## 12. 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 it is 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.

## 13. 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 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 boreholes, 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 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 locations.

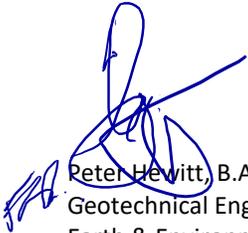
This report in no way reflects any on-site environmental considerations.

## 14. Closure

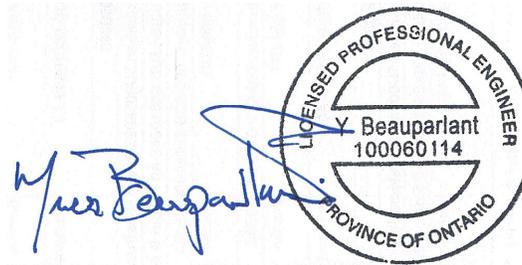
We trust that these comments provide you with sufficient information to proceed with design. Should you have any questions, please do not hesitate to contact this office.

Yours truly,

EXP Services Inc.



Peter Hewitt, B.A.Sc.  
Geotechnical Engineering Designer,  
Earth & Environmental  
Northeastern Ontario



Yves Beuparlant, P.Eng.  
Manager, Earth & Environmental  
Northeastern Ontario

## Appendix A – Drawings



KEYPLAN - N.T.S.

LEGEND

-  EXP BOREHOLE
-  TEMPORARY BENCHMARK  
(FINISHED FLOOR STORAGE HUT AT NORTH OF PROPERTY)

– NOTES –

- 1) 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.
- 2) Do not use Test Hole elevations for design purposes.
- 3) Soil samples will be retained in storage for 3 month and then destroyed unless client advises that an extended time period is required.
- 4) Quantities should not be established from the information provided at the Test Hole locations.
- 5) This drawing forms part of the report, project number as referenced, and should be used only in conjunction with this report.

Feb. 25, 2026 - 10:06am E:\SUD\SUD-26002251-A0\60\_Execution\62\_Reports\SUD-26002251-A0 - BOREHOLE LOCATION PLAN.dwg

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REVISIONS		
No.	DESCRIPTION	DATE

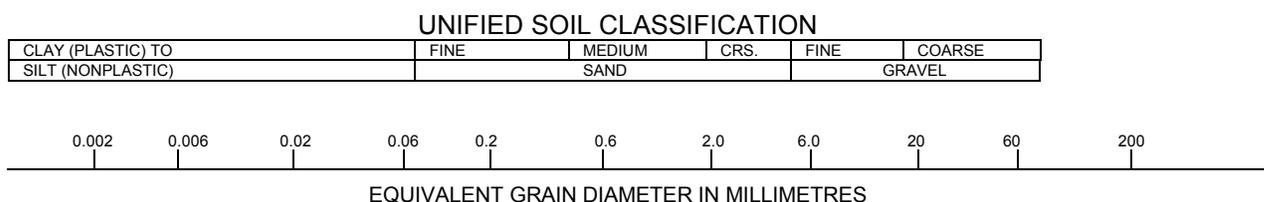
CLIENT	MUNICIPALITY OF CALLANDER
PROJECT	PROPOSED OPERATIONS FACILITY CALLANDER, ON
PROJECT NO.	SUD-26002251-A0

TITLE: BOREHOLE LOCATION PLAN		
DATE	SCALE:	DWG NO.
FEBRUARY 2026	NTS	A-1

## Appendix B – Borehole Logs

## Notes on Sample Descriptions

- 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.



**ISSMFE SOIL CLASSIFICATION**

CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		

- 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.
- 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 soundness 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-26002251-A0

Figure No. B-2

Project: Callander Operations Buildings

Sheet No. 1 of 1

Location: Callander, Ontario

626442, 5119435

Date Drilled: February 23, 2026

Auger Sample

Combustible Vapour Reading

SPT (N) Value

Natural Moisture

Dynamic Cone Test

Plastic and Liquid Limit

Shelby Tube

Undrained Triaxial at

Field Vane Test

% Strain at Failure

Penetrometer

Datum: Local (Non-Geodetic)

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	N Value				Combustible Vapour Reading (ppm)			SOIL SAMPLE	Sample Number
					20	40	60	80	25	50	75		
		<b>ASPHALT, 50 mm</b>	100.89	0									
		<b>FILL, sand, some gravel, trace silt, dark brown, moist, frozen</b>	100.8										AS1
		<b>FILL, sand, some silt, brown, moist, frozen</b>	100.3										
		<b>FILL, sand, some silt, brown, moist, frozen</b>	99.7	1									SS2
		<b>ROCK CHIPS</b>	99.4										SS3
		REFUSAL AT ~1.5 m ON SUSPECTED BEDROCK OR BOULDER											

SUDBURY GEO SUD-26002251-A0 CALLANDER OPS GPJ NEW GDT 3/4/26



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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

# Log of Borehole BH-2

Project No. SUD-26002251-A0

Figure No. B-3

Project: Callander Operations Buildings

Sheet No. 1 of 1

Location: Callander, Ontario

626453, 5119414

Date Drilled: February 24, 2026

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

Penetrometer

Drill Type: CME 75 Truck

Datum: Local (Non-Geodetic)

GWL	SYMBOL	Soil Description	ELEV. m	DEPTH	N Value				Combustible Vapour Reading (ppm)			SOIL SAMPLE	Sample Number	
					20	40	60	80	25	50	75			
		FILL, sand, some gravel, trace silt, dark brown, wet, frozen	100.90	0										
		FILL, sand, some silt, brown, wet, frozen	100.4											AS1
		>50/50 mm, frozen	99.8	1										SS2
		REFUSAL AT ~ 1.1 m ON SUSPECTED BEDROCK OR BOULDER												

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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-3

Project No. SUD-26002251-A0

Figure No. B-4

Project: Callander Operations Buildings

Sheet No. 1 of 1

Location: Callander, Ontario

626414, 5119389

Date Drilled: February 23, 2026

Auger Sample 

Combustible Vapour Reading 

SPT (N) Value 

Natural Moisture 

Drill Type: CME 75 Truck

Dynamic Cone Test 

Plastic and Liquid Limit 

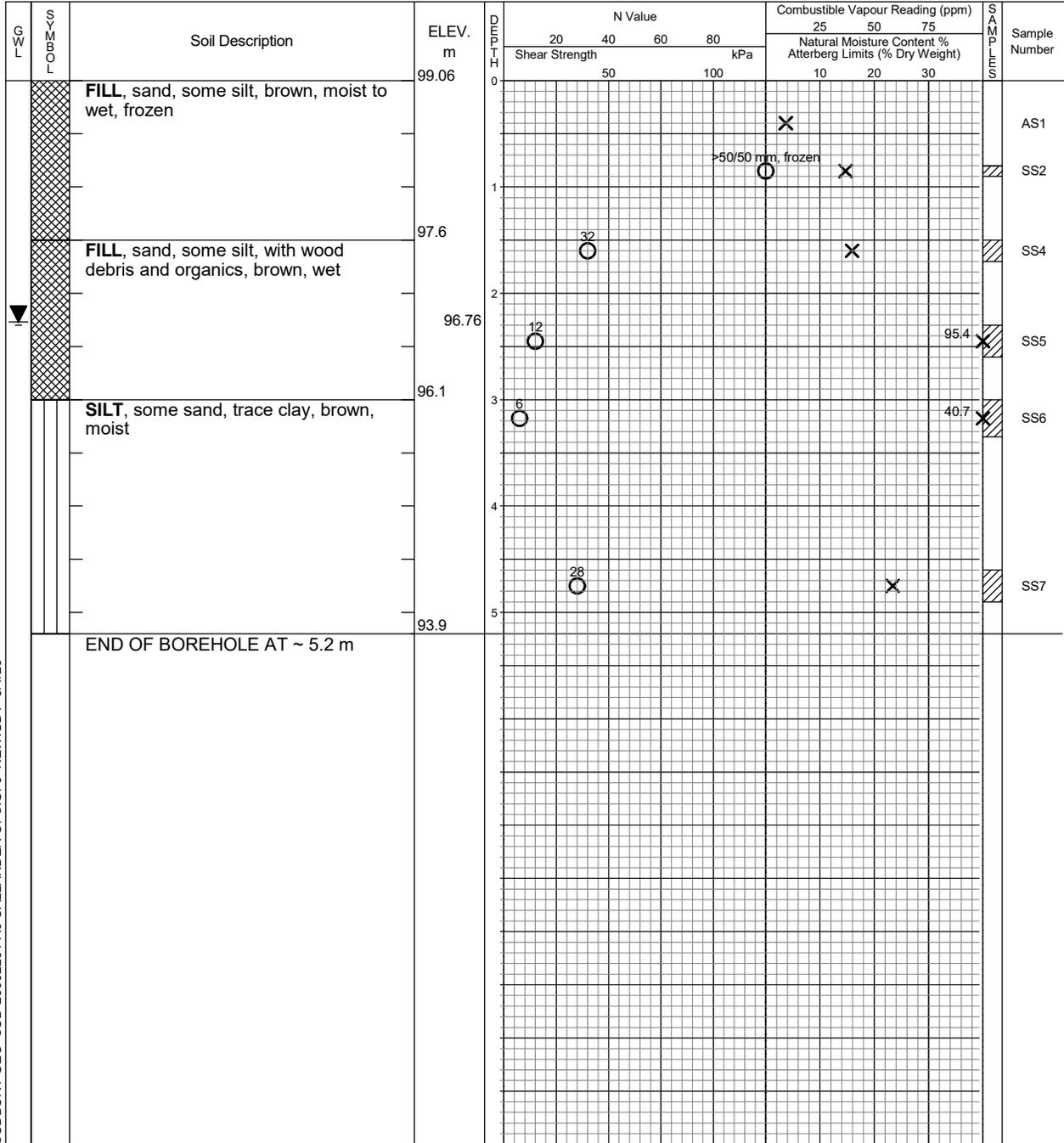
Datum: Local (Non-Geodetic)

Shelby Tube 

Undrained Triaxial at % Strain at Failure 

Field Vane Test 

Penetrometer 



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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	2.3	3.0

# Log of Borehole BH-4

Project No. SUD-26002251-A0

Figure No. B-5

Project: Callander Operations Buildings

Sheet No. 1 of 1

Location: Callander, Ontario

626424, 5119377

Date Drilled: February 23, 2026

Auger Sample 

Combustible Vapour Reading 

SPT (N) Value 

Natural Moisture 

Drill Type: CME 75 Truck

Dynamic Cone Test 

Plastic and Liquid Limit 

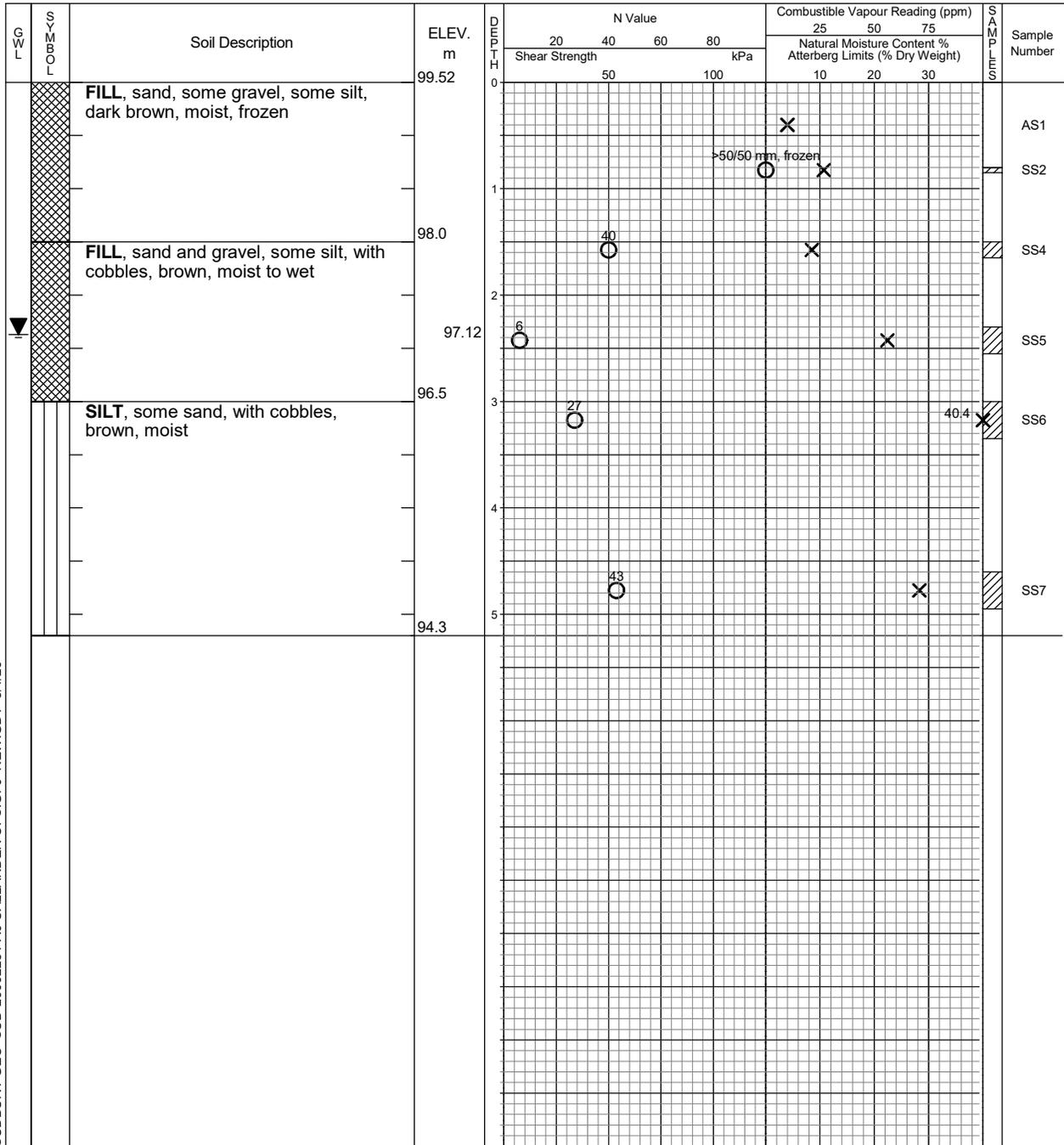
Datum: Local (Non-Geodetic)

Shelby Tube 

Undrained Triaxial at % Strain at Failure 

Field Vane Test 

Penetrometer 



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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

# Log of Borehole BH-5

Project No. SUD-26002251-A0

Figure No. B-6

Project: Callander Operations Buildings

Sheet No. 1 of 1

Location: Callander, Ontario

626434, 5119367

Date Drilled: February 23, 2026

Auger Sample

Combustible Vapour Reading

SPT (N) Value

Natural Moisture

Drill Type: CME 75 Truck

Dynamic Cone Test

Plastic and Liquid Limit

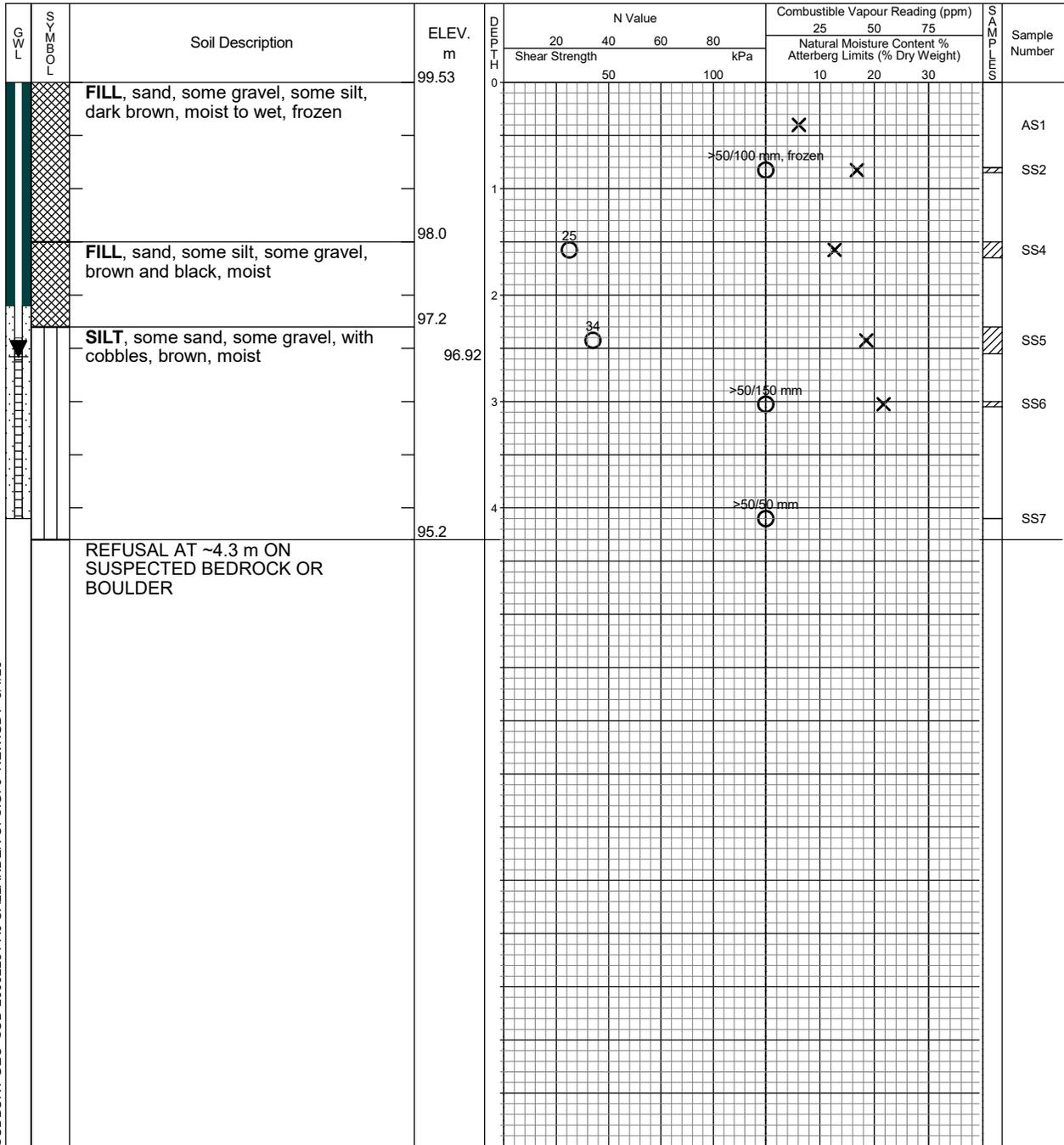
Datum: Local (Non-Geodetic)

Shelby Tube

Undrained Triaxial at % Strain at Failure

Field Vane Test

Penetrometer



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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	2.6	N/A
Feb 24, 2026	2.6	N/A
March 3, 2026	2.6	N/A