FINAL REPORT

Geotechnical Investigation

Proposed Building and Parking Lot Development, 250 Forestglade Crescent, Ottawa, Ontario

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1.0 INTRODUCTION

WSP Canada Inc. (WSP) was retained by OTTAWA ABORIGINAL COALITION c/o FOTENN to carry out a geotechnical investigation for the proposed residential building and parking lot development at a project site located at 250 Forestglade Crescent in Ottawa, Ontario. The approximate location of the project site is shown on the site plan included in this report as Figure 1. The geotechnical investigation and reporting were carried out in general accordance with the scope of work provided in WSP's proposal 2024CA314771 dated September 13, 2024, and approved on October 21, 2024.

The purpose of this investigation was to assess the general subsurface and groundwater conditions within the study area by means of four (4) boreholes and associated laboratory testing. The subsurface conditions obtained from the current investigation and available project details were used to prepare geotechnical recommendations for the geotechnical design aspects of the project, including construction considerations which could influence design decisions.

The reader is referred to the '*Important Information and Limitations of This Report'* which follows this report and forms an integral part of this document.

2.0 SITE AND PROJECT DESCRIPTIONS

The proposed development is located at 250 Forestglade Crescent in Ottawa, ON. The project site is bounded on the north by a bicycle path and existing houses, on the south by Forestglade Crescent, on the east by existing houses, and on the west by Blohm Drive. The site is currently vacant and covered with grass. Two mature trees are located on the east side near the proposed exterior parking lot. It is understood that the trees and associated root systems will be removed as part of the proposed building and parking lot development.

The site topography was relatively flat at the time of field investigation. The elevation at the northeast area was about 83.0 m above sea level (masl) and slightly dropped down to the south and west to the lowest elevation of approximately 82.0 masl. An existing underground hydro-one pipe was present northeast of the site.

As shown on the site plan on Figure 1, the proposed building will be located towards the west and center of the site while the proposed parking lot will be on the east end of the site. Based on the preliminary site development plans provided by FOTENN, it is understood that the proposed building will be a 2-storey residential development without below grade level (or basement) and cover an approximate footprint net area of 930 m^2. It is also understood that the proposed residential building will include offices, a multipurpose room, a dining area, a courtyard, a children's play area, and a garbage room. The proposed exterior parking lot will serve about 10 parking spots of about 9' by 18' each. The proposed finished site grade will generally be kept within about 0.5 m of the existing grade, and significant site grade is not anticipated as part of the proposed development.

The project will also include new utilities for water, storm, sanitary, electrical, and communications, which will be connected to the existing utilities.

3.0 DESKTOP REVIEW

3.1 Published Geological Information

Based on the physiography and surficial geology maps of Southern Ontario published by the Ontario Geological Survey (OGS), the project site is situated within the Clay Plains physiography but anticipated to be underlain by local older alluvial deposits consisting of clay, silt, sand and gravel, and may also contain organics.

Based on the bedrock geology map of Ontario published by OGS, the bedrock at the project site may consist of shale, limestone, dolostone, or siltstone.

4.0 METHOD OF INVESTIGATION

4.1 Field Investigation

The field work for the current geotechnical investigation was carried out on November 14, 2024, and included advancing a total of four boreholes (BH24-01 to BH24-04). The approximate borehole locations are shown on the site plan attached as Figure 1.

The boreholes were advanced with a CME-55 truck-mounted drill rig supplied and operated by George Downing Estate Drilling of Grenville-sur-la-Rouge, Quebec.

Soil samples were obtained using a 35 mm inside diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). Soil samples were obtained at vertical sampling intervals of about 0.76 m. Boreholes BH24-01 and BH24-02 were advanced to refusal on bedrock at depths of about 4.1 m (~EL. 79.1 m) and 4.4 m (~EL. 79.1 m) below the existing ground surface, respectively. Boreholes BH24-03 and BH24-04 were advanced to a depth of about 1.8 m below the existing ground surface (mbgs) or elevations of about 81.8 m and 81.7 m, respectively. Bedrock was cored at boreholes BH24-01 and BH24-02. NQ-sized bedrock core samples were obtained using a rotary diamond drilling technique up to a depth of about 5.7 mbgs or approximate elevations of 77.5 m and 77.7 m in BH24-01 and BH24-02, respectively.

Monitoring well was installed in borehole BH24-02 to allow for measurement of the groundwater levels in the soil overburden. The monitoring well consisted of a 51 mm inside diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill, and sealed by a section of bentonite hole plug. The groundwater level measurement in this well was carried out by WSP personnel on December 02, 2024.

Fieldwork was supervised by WSP's geotechnical staff who logged the boreholes, directed the in-situ testing, and collected the soil and rock samples retrieved in the boreholes. On completion of the drilling operations, the soil and core samples were transported to WSP's Ottawa laboratory for further examination by the project engineer and for laboratory testing.

The borehole coordinates and existing ground surface elevations were measured using a Trimble R10 GPS survey unit. The geodetic reference system used for the survey is the North American datum of 1983 (NAD83-CSRS). The borehole coordinates are based on the Universal Transverse Mercator (UTM Zone 18) coordinate system.

The borehole logs are included in Appendix A, and the rock core photos are included in Appendix B.

4.2 Laboratory Testing

Upon completion of the geotechnical field investigation, soil and rock core samples were transported to WSP's CCIL-certified laboratory in Ottawa, Ontario for further examination and testing. The following laboratory tests were carried out in general accordance with respective ASTM or applicable standards:

- Moisture content measurements (11 tests)
- Grain size distribution (3 tests)
- Atterberg Limits (2 tests)
- Basic chemical analyses for soluble sulphate, chloride, Electrical Conductivity, Resistivity, and pH (2 tests)

All tests were completed at WSP's Laboratory in Ottawa, except the basic chemical analyses which were completed by Eurofins Environment Testing. The laboratory test results are summarized in the borehole logs in Appendix A. The geotechnical test reports and basic chemical analysis reports are also included in Appendix C and D, respectively.

5.0 SUBSURFACE CONDITIONS

5.1 General

The Record of Borehole sheets in Appendix A describes the subsurface conditions at the boreholes' locations only. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress as well as results of Standard Penetration Tests and, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface soil, bedrock, and groundwater conditions will vary between and beyond the boreholes' locations.

5.2 Summary of Subsurface Stratigraphy

Based on the results of the borehole investigation, the general subsurface stratigraphy at the proposed building consists of topsoil over existing fill which is subsequently underlain by native clay, sand and gravel (till), and bedrock. The general subsurface stratigraphy at the proposed parking lot also consists of topsoil over existing fill that is underlain by silty sand with gravel to gravelly, which could be either a native or potential fill deposit.

Further descriptions of the soil and bedrock layers are provided in the subsections below.

5.2.1 Topsoil

Topsoil was encountered at the ground surface of all boreholes BH24-01 to BH24-04. The topsoil thickness ranged between 130 and 180 mm.

5.2.2 Existing Fill

Existing fill was encountered below the topsoil in all boreholes BH24-01 to BH24-04.

Cohesionless fills consisting of gravelly sand were encountered below the topsoil in BH24-02 and BH24-04. The fill in BH24-02 contained organics. The cohesionless fill at these borehole locations extended to depths ranging from about 0.4 to 0.9 mbgs (~El. 83.1 to 82.7 masl). The SPT blow count 'N' values in the cohesionless fills ranged from 10 to 20 blows per 0.3 m of penetration, which indicate compact density. A moisture content measured on one (1) selected sample of cohesionless fill was 8%.

Cohesive fills consisting of clayey silt were encountered below the topsoil in boreholes BH24-01 and BH24-03. The fill in BH24-01 contained organics. The cohesive fills extend to depths ranging from about 0.6 to 1.5 mbgs (~El. 83.0 to 81.7 masl). The SPT blow count ‘N’ values in the cohesive fills ranged from 12 to 26 blows per 0.3 m of penetration, indicating stiff to very stiff consistency. A moisture content measured on one (1) selected sample of cohesive fill was 10%.

5.2.3 Silty Sand with Gravel to Gravelly (Potential Fill)

A deposit of silty sand with gravel to gravelly was encountered below the fill layer in boreholes BH24-03 and BH24-04. The deposit also contained cobbles and boulders or rock fragments and may potentially be fill deposit as well. The silty sand deposit was encountered at depths of 0.6 to 0.9 mbgs (~El. 83.0 and 82.7 m) and extended to the borehole termination depths of 1.8 mbgs (~El. 81.7 and 81.8 m).

The SPT blow count ‘N’ values in the silty sand with gravel deposit ranged from 10 to 32 blows per 0.3 m of penetration, which indicate compact to dense density.

The moisture content measured on three (3) selected samples of silty sand with gravel deposit from boreholes BH24-03 and BH24-04 ranged between 7% and 12%. The results of the grain size distribution test carried out on two (2) selected samples were 25 and 26% gravel, 42 and 46% sand, and 33 and 28% fines, respectively. The reports of grain size distribution results are presented on Figures 3 and 4 in Appendix C.

5.2.4 Native Clay (Potential Sensitive Leda Clay)

At the location of boreholes BH24-01 and BH24-02, the fill was underlain by medium to high plastic clay with varying amounts of sand and gravel. The native clay layer may potentially be sensitive Leda clay. The clay layer was encountered at depths of 0.4 to 1.5 mbgs (~El. 83.1 to 81.7 masl) and extended to depths of 3.8 to 4.1 mbgs (~El. 79.7 to 79.2 masl).

The SPT blow count ‘N’ values in the native clay deposit ranged from 3 to 15 blows per 0.3 m of penetration, which indicate soft to stiff consistency.

The moisture content measured on six (6) selected samples of the clay layer from boreholes BH24-01 and BH24-02 ranged between 9% and 53%. The result of the grain size distribution test carried out on one (1) selected sample of the native clay deposit was 12% gravel, 23% sand, 23% silt, and 42% clay. The report of grain size distribution result is presented on Figure 2 in Appendix C. Atterberg limits tests completed on two selected samples of native clay gave a liquid limit of 34 and 58, the plastic limit of 19 and 25, and a plasticity index of 15 and 33, which indicate medium plastic (CI) to high plastic (CH) clay. The reports of the Atterberg limits test are presented on Figure 5 in Appendix C.

5.2.5 Sand and Gravel Glacial Till

Sand and gravel glacial till layer was encountered beneath the high plastic clay deposit in borehole BH24-02. The glacial till is characterized as a heterogeneous mixture of sand and gravel with cobbles, boulders and rock fragments. The glacial till layer was encountered at a depth of 3.8 mbgs (~El. 79.7 m) and extended to a depth of 4.4 mbgs (~El. 79.1 m).

The SPT blow count ‘N’ values in the glacial till was 26 blows per 0.3 m of penetration, indicating a compact density.

The moisture content measured on one (1) selected sample of the glacial till was 4%.

5.2.6 Bedrock

Boreholes BH24-01 and BH24-02 were advanced to refusal at depths of about 4.1 mbgs (~EL. 79.1 m) and 4.4 mbgs (~EL. 79.1 m), respectively. The boreholes BH24-01 and BH24-02 were then cored using diamond drilling after the auger/spoon's refusal to confirm bedrock. The bedrock coring was extended to a depth of about 5.7 mbgs (~El. 77.5 m and 77.7 m). It should be noted that the above bedrock depths were measured only at the locations of BH24-01 and BH24-02, and local deviation/variation of bedrock depths should be expected outside of the borehole locations.

Table 1 summarizes the ground surface elevations, depths and elevations to the top of glacial till and bedrock in the boreholes, as well as the termination depths and elevations of the boreholes. The depths and elevations do not include allowance for potential surficial loose or weak layers of glacial till or bedrock.

Table 1: Summary of Depths/Elevations to Top of Till and Bedrock, and Bottom of Borehole

Borehole Number	Ground Surface Elevation (masl)	Depth to Top of Till (m)	Elevation to Top of Till (m)	Depth to Top of Bedrock (m)	Elevation to Top of Bedrock (m)	Core Length (m)	Depth to Bottom of Borehole (m)	Elevation to Bottom of Borehole (m)
BH24-01	83.2	-	-	4.1	79.1	1.6	5.7	77.5
BH24-02	83.5	3.8	79.7	4.4	79.1	1.4	5.7	77.7

The bedrock encountered in the cored boreholes (BH24-01 and BH24-02) generally consists of moderately to slightly weathered, fine-grained, none to slightly porous, very thinly to medium bedded, grey Shale. Highly weathered and fragmented rock was encountered at borehole BH24-02 in the upper portion of the bedrock layer. The weathered bedrock layer extended to about 0.55 m from the top of bedrock. Photographs of the recovered bedrock cores are presented on Figures B1 to B4 in Appendix B.

Excluding the upper weathered bedrock layer in BH24-02, the Total Core Recovery (TCR) was 100%. The Solid Core Recovery (SCR) ranged between about 62 and 84%. The RQD values of the rock cores ranged between about 41 and 55%. The measured RQD values indicate fair to poor rock quality. Three rock core samples were attempted for unconfined Compressive Strength (UCS) testing; however, all selected samples were weak and broken during test preparation.

5.3 Groundwater Condition

The groundwater level in the monitoring well installed in borehole BH24-02 was measured on December 2, 2024. The measured groundwater level is summarized in Table 2.

Table 2: Summary of Groundwater Level Measurement in Monitoring Well

Borehole No.	Screen Zone	Groundwater Level Measurement on December 02, 2024 (Monitoring Well Installed on November 14, 2024)	
		Depth (m)	Elevation (masl)
BH24-02	Clay, and sand and gravel till soil	4.1	79.40

It should be noted that groundwater levels are expected to fluctuate seasonally. Higher groundwater levels or shallow perched water are expected during wet periods of the year, such as spring and fall.

5.4 Basic Chemical Analyses

Two (2) samples of soil from boreholes BH24-01 and BH24-02 were submitted to Eurofins Environment Testing for basic chemical analysis related to potential sulphate attack on buried concrete elements and corrosion of buried ferrous elements. The results of this testing are provided in Appendix D and are summarized below in Table 3.

Table 3: Summary of Basic Chemical Analyses Results

Borehole Number	Sample Number	Depth Intervals (m)	Chlorides (%)	Sulphates (%)	pH	Electrical Conductivity (mS/cm)	Resistivity (Ohm-cm)
BH24-01	3	1.5 – 2.1	<0.002	0.23	7.55	1.08	926
BH24-02	3	1.2 – 1.8	0.003	0.11	7.67	0.91	1,099

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical recommendations and comments for the design of the proposed development including a 2-storey building without basement, parking lot, and underground utilities based on our interpretation of the subsurface information and the project requirements.

The information in this portion of the report is provided for the geotechnical planning and design purposes by the design engineers. Where comments are made on construction, they are provided only to highlight aspects of construction which could affect the design of the project. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the factual information for construction, and make their own interpretation of the factual data as it affects their proposed construction techniques, costs, sequences, schedules, equipment and other resource requirements, and safety.

The geotechnical recommendations herein are provided in general conformance with the requirements of the National Building Code of Canada 2020 (NBCC 2020) and the latest online version of the Ontario Building Code (OBC).

6.2 Seismic Design

6.2.1 Seismic Site Classification

Based on Table 4.1.8.4.A. of OBC (latest online version) and Table 4.1.8.4.-B of NBCC (2020), a **Site Class D** can be considered for the firm to stiff cohesive soil and compact cohesionless soil overburden at the project site.

A Site Class C may be considered if the proposed building foundations are placed directly on the shallow bedrock stratum, or the existing overburden soils underneath the foundations are replaced with well compacted granular engineered fill or CEMATRIX Cellular concrete fill, or other equivalent approved backfill products.

6.2.2 Seismic Hazard Values

Seismic hazard values for the proposed project site were obtained from Natural Resources Canada (2020 National Building Code of Canada Seismic Hazard Tool – https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php).

The seismic hazard values for 2% probability of exceedance in 50 years are presented below:

- Peak Ground Acceleration (PGA): 0.390g for Site Class D, where g is 9.81 m/s^2.
- 5%-damped Spectral acceleration for period of 0.2 seconds (Sa (0.2)): 0.662g for Site Class D.
- Peak Ground Velocity (PGV): 0.369 m/s for Site Class D.

Seismic hazard values for 5%- and 10%-in-50-year, and Sa(T) values for different values of T can be obtained from the online seismic hazard tool referenced above.

6.3 Frost Protection

6.3.1 Frost Penetration Depth

The frost penetration depth at the project site is estimated to be 1.8 m based on Ontario Provincial Standard Drawing (OPSD) 3090.101.

6.3.2 Frost Cover Requirements

The upper existing fill and native clay layers at the project site are considered to be frost susceptible. Therefore, all unheated and partially heated foundation elements (including exterior side slabs and footings) should be protected against frost heave by providing a minimum of 1.8 m soil cover or with the use of rigid insulation.

6.3.3 Rigid Insulation Requirements

6.3.3.1 Insulation for Footings

In accordance with Canadian Foundation Engineering Manual (CFEM 2023), insulated footings should have a minimum depth of 0.76 m below the finished exterior grade. A minimum of 75 mm thick rigid insulation should be used to insulate the foundation wall extending from 0.3 m above the ground surface to the top of the footing pad and then extending horizontally to a minimum length of 1.2 m for partially heated footings and 2.4 m for unheated footings. For footing depths deeper than 0.76 m, the horizontal extent of insulation may be reduced in linear interpolation so that insulation will not be required at depth of 1.8 m. For example, if the footing depth is 1.28 m (halfway between 0.76 m and 1.8 m), then the insulation should extend horizontally to a minimum length of 0.6 m for partially heated footings and 1.2 m for unheated footings. The horizontally extended insulation should have a minimum soil cover of 0.3 m.

6.3.3.2 Insulation for Slab-On-Grade

Based on CFEM (2023), insulation of unheated slab should consist of a minimum of 100 mm thick rigid insulation placed underneath the slab to cover the entire slab footprint and extend horizontally at least 1.8 m beyond the edges of the slab. The rigid insulation should also be placed on a minimum of 200 mm thick, clean well graded granular engineered fill of max particle size 26 mm (such as Granular A). The rigid insulation extending outside the slab footprint should have a minimum soil cover of 300 mm. Partially heated slabs, such as along exterior perimeter of heated building, should also be provided with a rigid insulation underneath the slab extending at least 1.8 m inside and outside the slab from its edges. In addition, the rigid insulation should be designed to resist the compressive stress applied from the slab. The selected rigid insulation should have the required compressive strength to resist the load from the slab, and the insulation thickness may also need to increase depending on the applied compressive stress.

6.3.3.3 Insulation for Underground Utilities

Proposed underground utilities that will be affected by freezing such as watermain and sanitary should also be provided with a minimum of 75 mm thick rigid insulation if they are placed above the estimated frost depth of 1.8 m below the finished grade. The rigid insulation should extend horizontally over the utility pipe and vertically on both sides of the pipe. The vertical side insulations should extend to the depth of 1.8 m below the finished grade. The horizontal insulation over the pipe should have a minimum soil cover of 300 mm.

6.3.4 Frost Protection of Grade Beams and Pile Caps

Grade beams and pile caps (if any) should be protected from frost heave by providing a void form or crawl space between their undersides and the soil. The minimum thickness of void space should be 200 mm. A biodegradable void form should be considered to degrade over time following the placement of the concrete. Its strength must be sufficient to support the fresh concrete during construction. Alternatively, an engineered compressible medium (i.e., GeoSpan® or other approved manufacturer) may be used in lieu of a void-forming material, and the uplift pressures may be taken as the crushing strength of the compressible medium for 200 mm of deformation.

The backfill around the grade beams and pile caps should consist of non-frost susceptible materials. The finished grade adjacent to grade beams and pile caps should be capped with well compacted low plastic clay and sloped away so that the surface runoff is not allowed to accumulate in the void space or compressible medium. If water is allowed to accumulate in the void space or the compressible medium becomes saturated, the beneficial effect will be negated, and frost heaving pressures will apply on the grade beams or pile caps.

6.3.5 Construction Considerations for Frost Protection

If foundations are to be constructed during the winter months, foundation soils, especially the potential sensitive native clay layers, must be protected from freezing temperatures using suitable construction techniques. Therefore, the base of all excavations should be insulated such as using insulation tarps or provided with heat immediately upon exposure of freezing temperature, until the time that heat can be supplied to the building interior and/or the foundations have sufficient earth cover or permanent insulation to prevent freezing of the subgrade soils.

6.4 Soil Swell/Shrinkage Protection

Based on the liquid limit and plasticity index values measured in representative soil samples, the swelling/shrinkage potential of the native clay soil near the surface is considered to be high if the native soils have fluctuating moisture conditions such as access to water or drying conditions. The amount of swell and corresponding upward movement will depend on the availability of free water, detailed mineral composition of the clay particles, and loading conditions exerted by the structures.

The upper, medium to high plastic clay layer extends to the depth of about 4 mbgs at the borehole locations. The subexcavation and replacement of the high plastic clay layer with granular or other suitable engineered fill will be costly approach to mitigate the soil swelling/shrinkage issue.

As alternative approach, the in-situ moisture contents or conditions of the high plastic clay should be kept or not disturbed from its natural equilibrium state to reduce the chance of soil swelling and shrinkage. The high plastic soil should not be prone to excessive wetting or drying. Positive site drainage should be provided around all the structures to control surface water drainage away from the structures to keep the natural moisture condition of the high plastic soil. In addition, a layer of polyethylene sheeting 150 μm (minimum) thick should be placed underneath the concrete slabs above the granular bedding to deter migration of moisture into the subsurface soil.

The ground surface behind foundation walls should also be capped with minimum 300 mm thick low plastic soil or covered with impermeable polyethylene sheets to prevent the infiltration of surface water into the clay layer behind the foundation walls and reduced the effect of soil swelling on the retaining walls.

It should be noted that the above moisture controlling method, in lieu of clay removal and replacement approach, will have some level of soil swell risk as the moisture control measures (such as site drainage) may not be effective through time.

6.4.1 New Tree Planting and Existing Tree Management

The native medium to high plastic clay present at the project site is typically sensitive to moisture changes (causing settlement) due to the water demand by the planted trees. The selection of tree types and their planting should follow the City of Ottawa Guidelines (2017) for Tree Planting in Sensitive Marine Clay Soils. In general, the tree must be low water demand and planted at a minimum setback distance equivalent to the full mature height of the tree from a building foundation or structure.

6.5 Site Preparation and Grading Restriction

As discussed in the previous sections of this report, the subsurface stratigraphy generally consists of topsoil over existing fill which is underlain subsequently by medium to high plastic (potentially sensitive) native clay, sand and gravel glacial till, and shale bedrock. The project site is currently covered with grasses and few large trees around the site boundaries.

As part of the site preparation, all the topsoil, existing fills containing organics and rootlets, and other unsuitable materials should be removed from the footprint of the proposed project site. The exposed subgrade should be protected from disturbance of construction traffic and graded to quickly drain away surficial runoff from the project site.

If engineered backfill is required, then all the existing topsoil, organics, existing fills and soft or weak native soils should be removed to competent subgrade level prior to any planned engineered backfill underneath the proposed structures. The exposed subgrade after the removal of the above materials should be inspected and approved by a qualified geotechnical consultant prior to placement of engineered backfill.

6.5.1 Grade Raise Restriction

It is understood that the finished site grade and ground slab level will not be raised more than about 0.5 m above the existing grade as part of the proposed development. Due to the presence of potential sensitive native Leda clay at the project site, it is recommended that the finished grade should not be raised more than about 0.5 m above the existing ground surface level, particularly if shallow footings placed on the native clay are selected to support the proposed building. The combined pressures from the shallow footings and additional grade raise applied on the sensitive Leda clay may exceed the maximum past load history (i.e. Pre-consolidation pressure) of the sensitive clay and may cause excessive clay settlements.

6.6 Temporary Excavation, Dewatering, and Engineering Fill

6.6.1 Temporary Excavation

The proposed development will require excavations for the main building foundations, elevator pit, and parking lot area. The required excavations may extend up to the bedrock which was encountered at depths of 4.1 to 4.4 mbgs in the boreholes. A competent bedrock was encountered at a depth of 5 mbgs in BH24-02. The excavation may consist of the removal of unsuitable materials including topsoil, existing fill with organics and rootlets, as well as excavation of the native high to medium plastic sensitive clay and glacial till soils (including cobbles, boulders and rock fragments). In addition, deeper boring or coring through the bedrock may be required if rock-socketed pile foundations are considered for the proposed building. The bedrock surface profile may also fluctuate over the foundation excavation area and some minor stripping of the bedrock surface may be required for placement of shallow foundations.

The contractor should consider a suitable means and methods for excavation not to cause disturbance to the potential sensitive Leda clay.

Environmental excess soil characterization study should be completed for handling of the excavated materials and potential contaminations.

6.6.1.1 Excavations in Overburden

Excavations of overburden materials are anticipated to be handled using conventional hydraulic excavating equipment. Cobbles, boulders and rock fragments should be expected in the existing fill layer, and native clay deposit and glacial till layer. The proposed excavation means and method by the contractor should consider the removal requirement of these boulders and rock fragments.

As a minimum requirement, all side slopes of temporary open-cut excavations should conform to the Occupational Health and Safety Act (OHSA) – Regulation for Construction Projects (O. Reg. 213/91). The existing fill soil and native soils would be classified as Type 4 soils, and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements. Large size boulders and cobbles at the excavation side slope faces should be removed for worker safety. Stockpiling and equipment operating should be avoided from the excavation edge to a distance at least equal to the excavation depth to reduce instability of unsupported excavation slopes.

Where the available space limits the above cutback slopes, the temporary excavations can be advanced at steeper slopes and protected by temporary shoring systems designed and installed by the contractor. In addition to the above minimum requirement, all temporary excavations more than 2 m in depth should be properly designed by the contractor.

All **permanent** excavations should be properly designed by a geotechnical consultant.

6.6.1.2 Excavations in Bedrock

Based on the current field investigation, the bedrock at the project site consists of mostly moderately to slightly weathered, bedded, grey shale bedrock at shallow depths of 4.1 to 4.4 mbgs (~El. 79.1 masl) in the boreholes.

The contractor is responsible for proposing suitable means and methods of rock excavation approved by the project owner. The proposed excavation method should enable control of the extent of excavation and should mitigate the potential risk of overbreak, unexpected over-excavation, and other rock disturbances or damage due to excavation.

The thin upper portion of the bedrock may be highly weathered. Thus, the shallow, localized bedrock excavation may potentially be carried out using mechanical excavating methods such as hydraulic breaker equipment (hoe ramming or percussion). However, more extensive bedrock excavation will be economical to be carried out using drill and blast techniques if needed. Closely spaced line drilling is typically used to control the extent of excavation and to reduce the potential for overbreak and unexpected over-excavation in both methods of mechanical excavation and blasting. For rock anchor and rock socketed pile constructions, rotary diamond coring may be required to advance the rock anchors and piles into the bedrock.

Vibrations induced by excavation activities will need to be considered when assessing potential impacts to adjacent structures. Caution should be exercised in carrying out bedrock removal around services and structures that may be sensitive to vibrations. Bedrock excavation should therefore be controlled to limit the peak particle velocities at all adjacent structures and services such that the risk of vibration-induced damage will be mitigated. A detailed rock excavation plan should be designed and prepared by a specialist in this field. The excavation plan should provide detailed information on the proposed excavation methods, vibration monitoring equipment, monitoring locations, frequency of readings, vibration limit criteria for reporting, suitable mitigation actions, and other relevant information. The excavation plan should be reviewed and accepted by the project owner and geotechnical consultant.

The rock excavation work should follow the general requirements in accordance with OPSS.MUNI 120 (November 2019) "General Specification for the use of Explosives". The recommended radius of the pre-excavation survey and maximum peak particle velocity values for different structures and services are included in Tables 1 and 2 of OPSS.MUNI 120 (November 2019). If unusually sensitive receptors are identified during construction planning, then specific criteria may need to be adopted for those receptors.

If practical, vibration intensive construction activities, such as rock excavations, should commence at the furthest points from sensitive receptor structures and services.

It is recommended that the monitoring of ground vibration intensities from the rock excavation operations be carried out both in the ground adjacent to the structures and within the structures themselves.

Vibration monitoring should be carried out throughout all bedrock excavation operations.

Contractor should provide a copy of the rock excavation/blasting plan and well in advance notice to the project owner. Contractor should also provide to the project owner copies of the pre-construction inspection, vibrating monitoring reports, and post-construction inspection reviews among any other relevant deliverables.

6.6.2 Temporary Dewatering

Based on the shallow groundwater condition observed in the installed monitoring well in BH24-02, a design groundwater level of 3.5 mbgs (~El. 80 m) is recommended considering the seasonal groundwater level fluctuations.

Excavations deeper than about 3.5 mbgs (~El. 80 m) may be under the groundwater, depending on the time of year that construction occurs. The rate of groundwater inflow into the excavations will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, the number of working areas being excavated at one time, and the time of year at which the excavation is made. Also, there may be instances where precipitation collects in an open excavation following rainfall and must be rapidly pumped out.

According to O.Reg 63/16 and O.Reg 387/04, if the volume of water to be pumped from excavations for the purpose of construction dewatering is greater than 50,000 litres per day and less than 400,000 litres per day, the water taking will need to be registered as a prescribed activity in the Environmental Activity and Sector Registry (EASR) and requires the completion of a "Water Taking Plan". A Permit to Take Water (PTTW) is required from the Ministry of the Environment Conservation and Parks (MECP) if a volume of water greater than 400,000 litres per day is to be pumped out from an excavation.

Temporary dewatering systems are the Contractor's responsibility, and the rate and volume required for dewatering is dependent on the construction methods and staging chosen by the contractor. A groundwater management plan should be submitted by the Contractor along with the study to determine the need of EASR registry and PTTW by MECP.

6.6.3 Engineered Fill

Structural engineered fill should be used for grade raise underneath load supporting structures such as footings and ground slabs. OPSS 1010 Granular A material may be considered as structural engineered fill. The granular structural engineered fill should be placed in maximum loose lifts of 200 mm and compacted to minimum 98% of Standard Proctor Maximum Dry Density (SPMDD) for the first lift and minimum 100% of SPMDD for the remaining lifts, at ±2% of Optimum Moisture Content (OMC). In tight space or restricted access areas where compaction will be difficult, ***or if the underlying native sensitive Leda clay is softer and prone to compaction disturbance***, then Cellular Concrete fill may be used to replace the granular engineered fill. The minimum required thickness, density and compressive strength of the Cellular Concrete fill should be designed to support the anticipated structure loads. The hydrostatic uplift pressure should also be considered in the design of the Cellular Concrete fill. The Cellular Concrete fill should be allowed to cure for some days prior to loading. The structural engineered fill (Granular A or Cellular Concrete fill) should extend laterally beyond the edge of load supporting structures, such as footings and slabs, to a minimum distance equal to the fill thickness or 1 m, whichever is greater.

For utility trench backfill, Unshrinkable Fill (U-Fill) as per OPSS.MUNI 1359 may be used in lieu of engineered backfill.

General engineered fill should be used for grade raise outside of the load supporting structures such as roads, sidewalks, and landscaping. The general engineered fill is not recommended to consist of the excavated existing fill soils containing organics and topsoil, as well as the high plastic native clay soil. Existing soils consisting of substantial silt contents should also be avoided to reduce frost heave issues. The general engineered fill may include imported low plastic clay fill or well graded granular fill such as Granular A or B Type I & II as per OPSS 1010. The imported fill should be free from topsoil, organics, debris, boulders, cobbles, rootlets and other unsuitable materials. The general engineered fill should be placed in maximum loose lifts of 200 mm and compacted to minimum 95% of Standard Proctor Maximum Dry Density (SPMDD) for the first lift and minimum 98% of SPMDD for the remaining lifts, at ±2% of Optimum Moisture Content (OMC).

The exposed subgrade should be inspected by qualified geotechnical consultant to confirm its stiffness and that the bearing surfaces have been adequately prepared and cleaned prior to the placement of the engineered fill. The prepared subgrade should be protected from disturbance of construction traffic, excessive wetting or drying.

6.7 Reuse of Existing Soils

The reuse of any excavated existing soils for construction backfilling is subjected to environmental assessment and suitability of the existing soils for backfill reuse purpose. Environmental assessment of the existing soils is not part of the scope of this report. WSP Environmental Services should be contacted to provide a Excess Soil Characterization Report (SCR) for the excavated excess soils and determine the environmental suitability of the excavated existing soils for reuse purpose.

6.8 Foundations

Based on the proposed two-storey building plan and encountered subsurface condition, the following types of foundations may be considered:

- Shallow spread and strip footings placed on native clay deposit
- Pile foundation socketed into bedrock
- Raft slab foundations

The native clay soil was stiff to soft in strength. The clay soil in BH24-02 was also found to be high plastic with very high water content in the depths of about 2.5 to 3.75 mbgs. Therefore, shallow footings supported on the native clay deposit may have low bearing capacity and may be prone to some settlements. Due to the presence of shallow bedrock at the project site, pile foundations socketed or embedded into the bedrock may also be considered as alternative foundation system if shallow footings are not able to support the proposed building loads. Raft slabs can also be considered as an alternative foundation system.

Supporting foundations on variable bearing strata (with different stiffnesses) should be avoided to limit the potential differential settlements between foundations. Supporting part of the foundations on glacial till or bedrock and other parts of the foundations on the native clay layer may induce differential settlement due to the compression of the upper clay layer compared to the glacial till or bedrock. Where possible, supporting part of the foundations on glacial till and the other part on bedrock should also be avoided to limit the differential settlement within the same structure.

6.8.1 Shallow Spread and Strip Footings on Native Clay

As discussed above, the medium to high plastic native clay deposit was stiff to soft in consistency and would provide low bearing capacity to support the shallow foundations. The recommended bearing resistances for the design of shallow footings supported on the undisturbed, native clay deposit are provided in Table 4.

Table 4: Recommended Bearing Resistances for Shallow Foundations

Bearing Stratum	Minimum Founding Depth (m)	Factored Ultimate Bearing Resistance at ULS (kPa)	Unfactored Bearing Resistance at SLS (kPa)
Competent Native Clay	1.5 m (1)	75 (2)	50 (3)

Notes:

(1) Footings should be placed on competent, undisturbed, native clay deposit; however, footings shall not be placed shallower than 1.5 m for bearing capacity requirement and to extend through the upper potential fill layer. Footings placed shallower than 1.8 mbgs should be provided with insulation as recommended in Section 6.3.

(2) A resistance factor of 0.5 was used to calculate the factored ultimate bearing resistance at ULS.

(3) The SLS bearing resistance was calculated for the corresponding estimated footing settlement of less than 25 mm.

Bearing resistances for footings subjected to significant eccentric and/or inclined loads should be assessed on case-by-case basis by WSP.

Where possible, footings should not be placed on different bearing strata. If footings are required to be supported on different strata for other reasons, then transitional taper zone should be designed and provided for gradual transition of bearing stratum stiffness from native clay to glacial till or bedrock. The taper zone inclination should be designed flatter than or equal to 4H:1V. The footings should also be designed structurally to accommodate the transition in bearing stratum stiffness.

The exposed native clay bearing stratum should be inspected by qualified geotechnical consultant to confirm its stiffness and that the bearing surfaces have been adequately prepared and cleaned.

The prepared subgrade should be protected from ingress of free water, frost, desiccation or drying, and disturbance due to construction traffic. The contractor should consider a suitable means and methods for excavation and construction of foundations without causing disturbance to the potential sensitive Leda clay. Footings must not be placed on fill, organic, disturbed, loose/soft, or frozen soils. Bearing soils which become frozen, dried, or softened should be removed and replaced with structural engineered fill as recommended in Section 6.6.3. In tight space or restricted access areas where compaction will be difficult, or if the underlying sensitive native clay is softer and prone to compaction disturbance, then Cellular Concrete fill may be considered in lieu of granular engineered fill.

6.8.1.1 Horizontal (sliding) Resistance of Footings

For cast-in-place concrete footings, resistance to horizontal loads (sliding resistance) can be calculated by considering the sliding friction resistance between the concrete footing base and the bearing stratum. The recommended interfacial adhesion strength between cast-in-place footing concrete and native clay soil bearing strata is provided below:

- Cast-in-place concrete footing to native clay soil: c_a = 30 kPa

The calculated sliding resistances using the above interface adhesion strength will be the ultimate value. A geotechnical resistance factor of 0.8 should be used to calculate the factored ultimate sliding resistance.

The resistance to horizontal loads could be increased by constructing a permanent passive resistance on the footing sides or constructing a shear key at the bottom of the footing if needed.

6.8.2 Rock Socketed Concrete Piles

If shallow footings are not able to support the proposed building loads (i.e. unpractical large footing sizes are required), then conventional cast-in-place concrete piles socketed into the bedrock may be considered as an alternative foundation option.

Rock socketed concrete piles may be designed using a recommended factored ultimate end bearing resistance of **1,500 kPa** at ULS. Settlement of rock socketed piles will not govern the design of the proposed building foundations. The concrete piles should be socketed to a minimum socket length of three times pile diameter (3B) or 1.5 m, whichever is greater, below the *competent bedrock surface*. The competent bedrock surface was encountered at a depth of 4.1 mbgs in BH24-01 and 5 mbgs in BH24-02. It should be noted that the competent bedrock surface depths will vary outside of the boreholes' locations across the site.

The rock socket bore or base shall be adequately cleaned of debris in order to rely on the socket end bearing resistance. The method of cleaning proposed by the selected piling contractor should be approved by a qualified geotechnical engineer prior to commencement of field works. The cleaned rock socket bore or base should be visually inspected by a qualified geotechnical engineer during construction. Should the inspection indicate that loosened material is present at the base, the base would need to be re-cleaned and re-inspected.

The factored geotechnical resistance of pile should also be limited to the structural capacity of pile calculated as the factored structural compressive strength of concrete multiplied by pile cross-section.

Piles should be spaced at center-to-center spacing of at least three times pile diameter (3B) to minimize pile group effect. A minimum shaft diameter of 400 mm is recommended for cast-in-place piles to minimize void formation during pouring of the concrete.

6.8.2.1 Cast-In-Place Pile Construction Considerations

Construction of cast-in-place rock socketed piles may be challenged by excessive soil sloughing and water seepage which will need temporary steel liners as well as dewatering (bailing water) or tremie method of construction. The potential presence of cobbles and boulders within the native soil deposit, especially in the glacial till, should also be considered in selecting pile boring equipment. Rotary diamond coring will be required to advance the rock socket bore in the bedrock. It is also required to pour concrete as soon as practical after drilling the holes in order to reduce the amount of soil sloughing and water seepage.

Provisions should be given for the presence of hard cobbles and boulders, and rock drilling/coring requirement in selecting suitable drilling equipment to advance pile boring through the obstructions, especially in the glacial till deposit, and through the bedrock.

Full-time monitoring of cast-in-place pile installation by a qualified geotechnical inspector is recommended to confirm the proper installation of piles and rock sockets or base cleaning.

6.8.3 Raft Slab Foundation and Slab-On-Grade

Raft slab foundation may also be considered as an alternative option to footings or piles. The raft slab foundations and slab-on-grade of the proposed building may be supported on the native competent clay stratum, or on structural engineered fill (described in Section 6.6.3).

6.8.3.1 Subgrade Preparation

All unsuitable materials such as topsoil, organics, existing fill, boulders, cobbles, and any wet, weak, or disturbed native clay soil should be stripped off from the proposed slab-on-grade and raft slab footprints. The exposed subgrades after excavation should be thoroughly cleaned of debris and loose materials. The excavated subgrade should also be visually inspected and approved by a qualified geotechnical consultant prior to placement of engineered fill or slab-on-grade or raft slab construction.

Any required grade raising of the excavated subgrade to the design slab subgrade level should consist of structural granular engineered fill recommended in Section 6.6.3. The structural granular engineered fills should extend laterally and connected to side drainage system to reduce local ponding of water inside the granular engineered fills. The exposed native clay stratum should be carefully reviewed to determine if compaction of the granular engineered fill will further disturb the native clay soil. An alternative grade raise structural backfill of Cellular Concrete fill should also be considered in the design stage. The Cellular Concrete fill should be structurally designed to resist the loads from the building and slab-on-grade or raft slab.

The prepared subgrade should be protected from disturbance of construction traffic, excessive wetting, or drying. The prepared subgrade should also be inspected and approved by a qualified geotechnical consultant prior to the installation of granular bedding and concrete slab.

6.8.3.2 Granular Bedding

A minimum of 200 mm thick, clean well-graded crushed stone granular bedding (grain size distribution satisfying OPSS.MUNI 1010 Granular A with less than 5% of fine particles passing 75μm sieve) should be installed on the prepared subgrade for the purpose of leveling and draining. The granular bedding should be installed in a single lift and compacted to 100% of SPMDD at ±2% of OMC.

6.8.3.3 Vertical Modulus of Subgrade Reaction

Load bearing slab-on-grade or raft slab should be structurally designed. The recommended vertical modulus of subgrade reaction values for the design of slab-on-grade or raft slab are provided below for anticipated bearing strata:

- Native clay soil bearing stratum: 3/B (MPa/m)

Where B (in meters) is the shortest dimension of the loaded area on slab-on-grade or raft slab. The slab design should consider the variability of stiffness between native soil, engineered fill, and bedrock bearing strata. As discussed previously, taper zones should be designed and provided for the gradual transition of bearing strata stiffness from bedrock to native soil or engineered fill. The taper zone inclination should be designed flatter than or equal to 4H:1V. In addition, the slab-on-grade or raft slab should be designed structurally to accommodate the transition in bearing stratum stiffness. Expansion joints should also be provided for the slab-on-grade as required by the design.

6.8.3.4 Permanent Drainage

It is understood that the proposed building will not include a below grade level (or basement).

As discussed in Section 6.6.2, a design groundwater level of 3.5 mbgs (~El. 80 m) is recommended based on the water level measurement from the installed monitoring well and considering the seasonal groundwater level fluctuations. Therefore, the proposed building ground floor slab and/or raft slab and foundation walls are anticipated to be above the groundwater level and will not be subjected to hydrostatic pressures.

However, the prepared subgrade within and around the proposed building footprint may consist of different soil materials including existing fill, native clay soil, granular engineered fill, and bedrock. Thus, the different subgrade soil conditions may block the free movement of water for draining and cause local ponding of water beneath the building footprint. In addition, the shallow shale bedrock may have relatively lower permeability, except at the weathered and/or fractured zones, and may hinder the infiltration of surface water and perched water into the ground.

A perimeter weeping tile subdrain system should be provided around the proposed building footprint to facilitate drainage of surface water infiltration and perched water away from the building foundations and ground slabs. The weeping tile subdrain system may consist of perforated pipes surrounded by free drain granular material (OPSS.MUNI 1004 19 mm Clear Stone or approved equivalent) and wrapped up with OPSS.MUNI 1860 Class II non-woven geotextile (Terrafix 360R or approved equivalent). The subdrains should be connected to the site drainage system or catch basins. Alternately, they can be drained into a sump and pump out. Inspection and maintenance of the subdrain system are recommended to ensure that the drainage system does not become blocked. A schematic diagram of the perimeter drainage system is presented on Sketch 1 included in this report.

Due to the shallow groundwater table, the slab-on-grade or raft slab should be provided with impermeable damp-proof membranes, such as a minimum 150 μm thick polyethylene sheet vapor barrier.

The perimeter subdrain systems should be properly designed by the Civil Design Consultant of the project. The above recommendations are provided for general guidelines only.

6.9 Earth Retaining Structures

It is understood that the proposed building will not consist of a basement or below grade level. The below recommendations for lateral earth pressures are provided for any potential earth retaining structures proposed at the project site.

6.9.1 Lateral Earth Pressures

Lateral earth pressures will need to be considered in the design of earth retaining structures or walls. The lateral earth pressures will depend on retained soil type, backfill type and compaction method, surcharge loads, wall movement, seismic effect, and drainage condition.

For retaining walls that are designed to allow sufficient lateral movement, active earth pressure may be used for design. For rigidly tied and unyielding structures, such as the proposed building basement walls, the at-rest earth pressure should be used for design.

The recommended earth pressure coefficients are provided in Table 5 for static conditions and in Table 6 for seismic (static plus dynamic) conditions. The earth pressure coefficients were determined for the assumed conditions of no wall-to-soil friction, vertical back of the wall, and horizontal back slope of the ground surface behind the wall. The earth pressure coefficients for the retained soils along the active/passive failure planes should be used for lateral earth pressure calculations. The failure planes rise from horizontal at 45 + $\phi/2$ for the active pressure condition and 45 – $\phi/2$ for the passive pressure condition.

Table 5: Static Lateral Earth Pressure Coefficients

Material	Unit Weight (kN/m^3)	Effective Friction Angle (deg)	Coefficients of Static Lateral Earth Pressure		
			Active, Ka	At rest, Ko	Passive, Kp
Granular A or Granular B Type II	21	35	0.27	0.43	3.69
Existing Fill or Native Clay Soil	19	25	0.41	0.58	2.46
Native Glacial Till	21	33	0.29	0.46	3.39

Table 6: Seismic (Static + Dynamic) Lateral Earth Pressure Coefficients

Material	Seismic Earth Pressure Coefficients (Site Class D, 2% probability in 50 yrs)		
	Active, K_{AE} (Yielding)	Active, K_{AE} (Non-Yielding)	Passive, K_{PE}
Granular A or Granular B Type II	0.39	0.57	3.30
Existing Fill or Native Clay Soil	0.56	0.84	2.13
Native Glacial Till	0.42	0.61	3.01

The point of application of the active lateral seismic (static + dynamic) earth pressure should be calculated as follows:

- Static active lateral earth pressure acts at H/3 of the wall, measured from the base upwards; and
- Dynamic active lateral earth pressure acts at 0.6 H of the wall, also measured from the base upwards.

The location of the applied earth pressures described above has the effect of moving the point of application of the seismic pressure (which is the combined static and dynamic lateral earth pressures) closer to the mid-height of the wall. The above point of application is for lateral pressure due to the weight of retained soil (γh), and the calculated point of applications are presented in Table 7. For a uniform surcharge load at the top of backfill surface, the seismic pressure point of applications should be recalculated by considering a static pressure point of application of 0.5H for uniform surcharge load.

For higher walls, the point of application should be established from complex dynamic analysis methods.

Table 7: Load Application Height (h) from Base of Wall as a Ratio of Wall Height (H)

Material	h/H Ratio for Seismic (Static + Dynamic) (Site Class D, 2% probability in 50 yrs)		
	For Active, K_{AE} (Yielding)	For Active, K_{AE} (Non-Yielding)	For Passive, K_{PE}
Granular A or Granular B Type II	0.42	0.47	0.30
Existing Fill or Native Clay Soil	0.41	0.47	0.29
Native Glacial Till	0.41	0.47	0.30

6.9.2 Backfill Behind Earth Retaining Structures

Excavated sand and gravel soil from the project site may be considered for backfilling behind the earth retaining structures by separating and removing the boulder/cobble and silt/clay contents from the excavated soils. If additional backfill materials are required, non-frost susceptible, free-draining granular fill conforming to OPSS Granular A or Granular B (Type I or II) with a maximum particle size of 26.5 mm and less than 5% fines content (or other approved equivalent) should be used to reduce problems with frost adhesion and heaving. A Class II non-woven geotextile separator as per OPSS.MUNI 1860 should be placed between the existing soil and free-draining granular fill to filter fines from water.

To avoid ground settlements around the earth retaining structures, which could affect site grading and drainage, all of the backfill materials should be placed in maximum loose lifts of 200 mm and compacted to at least 95% of SPMDD at ±2% of OMC. Care must be taken during the compaction operation not to overstress the retaining structures. Heavy construction equipment should be maintained at a distance of at least 1 m away from the retaining structures while the backfill soils are being placed, and the backfill should be uniformly raised around the retaining structures. Hand operated vibratory compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the earth retaining structures.

In areas where pavement or other hard surfacing will abut the proposed earth retaining structures, differential frost heaving could occur between the granular fill and the adjacent areas. To reduce this differential heaving, the backfill adjacent to the retaining structures should be placed to form a frost taper. The frost taper should be brought up to a pavement or other hard surface subgrade level from 1.8 m below finished exterior grade at a slope of 4H:1V or flatter, away from the retaining structures.

6.9.3 Drainage Behind Earth Retaining Structures

Drainage behind earth retaining structures should be provided by means of perforated pipe subdrains at the bottom of retaining structures. The perforated pipes should be surrounded by free drain granular material (OPSS.MUNI 1004 19 mm Clear Stone or approved equivalent) and wrapped up with OPSS.MUNI 1860 Class II non-woven geotextile. The pipes should be directed by gravity drainage to nearby storm sewer or sump pit.

The top of free-draining granular backfill behind the earth retaining structures should be sealed with either an impermeable geomembrane or a 300 mm clay layer.

The retaining structures should also be provided with impermeable damp-proof membranes.

The drainage system should be designed by a qualified Civil Design Consultant of the project to effectively mitigate the buildup of hydrostatic pressure behind the retaining structures. The subdrain pipes should be inspected and maintained to ensure that they do not become blocked.

6.10 Underground Utility Services

6.10.1 Trench Excavation and Subgrade Preparation

The site subsurface stratigraphy within the typical range of depths of underground utility pipes consists of the existing fill, native clay, glacial till and bedrock. The design groundwater level is recommended at 3.5 mbgs (~El. 80 m) as discussed in Section 6.6.2. Perched water seepage may also be encountered at shallower depths in the excavations during construction.

Utility trench excavations and temporary dewatering should be carried out in accordance with the general recommendations provided in Section 6.6 of this report.

The proposed bedrock excavation into the slightly to moderately weathered shale may be challenging and should be considered by the contractor in the selection of suitable method of excavation and equipment. The rock excavation in utility trenches should also follow the specifications in OPSS.MUNI 403.

Where the new underground utility alignments are supported on overburden soil, all the topsoil, organics, rootlets, boulder, cobbles and existing fill containing deleterious materials should be sub-excavated and removed from the utility trenches. The exposed trench subgrade should be visually inspected and weak subgrade areas should be further sub-excavated. The sub-excavated trench subgrade should be restored to the design utility grade level using native sand and gravel soil (excluding boulders and cobbles) or granular engineered fill compacted to at least 97% of SPMDD at ±2% of OMC.

The exposed subgrade after trench excavation should be properly cleaned off debris and loose materials. The prepared trench subgrade after excavation or engineered fill replacement of unsuitable materials should be inspected and approved by a qualified geotechnical consultant prior to placement of bedding layer and utility installation. The prepared subgrade should also be protected from disturbance of construction traffic, excessive wetting or drying, or freezing. No more than 15 m of trench should be open in advance of the completed utility installation.

6.10.2 Utility Trench Backfilling

In general, the installation of pipe utilities such as sanitary, storm and watermain should be completed in accordance with OPSS.MUNI 401 and OPSD 802 series drawings for the applicable pipe type and subgrade type. Where applicable, the Manufacturer’s specifications should be followed for the installation of utilities including gas, electrical and communication.

The bedding and cover materials for rigid pipes, the embedment material for flexible pipes, and the backfill material above the cover or embedment are specified in Section 401.05 of OPSS.MUNI 401. The minimum required thickness and extent of the bedding and cover materials or embedment material should be referenced from the applicable OPSD 802 drawings.

Where specifications are not available, at least 150 mm of Granular A (OPSS.MUNI 1010) should be used for bedding. The bedding material should extend to the spring line of the utility pipe or conduit. Cover material, from the top of bedding to at least 300 mm above the top of the utility, should consist of Granular A or Granular B (Type I or II) material with a maximum particle size of 26.5 mm.

The use of clear crushed stone as a bedding layer should not be permitted anywhere on this project since fine particles from the native soils could potentially migrate into the voids in the clear crushed stone and cause loss of lateral pipe support.

All the bedding, cover and embedment materials should be placed in maximum loose lifts of 200 mm and compacted to at least 95% of the material's SPMDD at ±2% of OMC. The materials should be placed on each side of the utility pipe or conduit simultaneously. At no time should the material levels on each side the pipe or conduit differ by more than 200 mm of uncompacted layer. Additional requirements should be followed in OPSS.MUNI 501.

Where compactions of engineered fill potentially disturb the native sensitive clay layer, Unshrinkable Fill (U-Fill) as per OPSS.MUNI 1359 may be used for trench backfill in lieu of the engineered soil backfill as discussed in Section 6.6.3.

It may be possible to re-use the excavated inorganic (excluding topsoil and organics) native soils as trench backfill above the cover or embedment fills, but the native soils should be carefully separated during excavation. Wet soil, cobbles, and boulders should be separated from the native soils. Wet native soils may be allowed to dry to an appropriate water content for placement and compaction. The separated native soils should be inspected and approved by a qualified material consultant prior to use as backfill. Suitable engineered fill could be imported if additional material is required to backfill the trench above the cover or embedment fills. Where the trench will support structures such as pavements and sidewalks, it is recommended to use granular engineered backfill material to reduce the potential trench backfill settlement. The granular backfill material should be placed to form frost taper on the trench sides. The trench backfill above the cover or embedment fills should be placed in maximum loose lifts of 200 mm and compacted to at least 97% of SPMDD at ±2% of OMC.

6.11 Asphalt Pavements and Concrete Aprons

It is understood that the proposed development will also consist of outdoor or exterior parking lot on the east side of the project site. The proposed exterior parking lot will serve about 10 parking spots of about 9' by 18' each. In addition, concrete sidewalks for pedestrians and concrete apron for garbage/dump truck are expected as part of the exterior development.

6.11.1 Subgrade Preparation

All unsuitable materials including topsoil, organics, existing fill containing organics or debris, and weak or soft subgrade soils should be sub-excavated and removed from the footprints of the proposed asphalt pavements and concrete aprons or sidewalks. The subcut of unsuitable materials is recommended to be conducted as a final stage after the construction of the proposed building and site utility services to reduce the amount of disturbance due to construction traffic and surface water.

It should be noted that the topsoil and existing fill contents and thicknesses presented in the borehole logs are only at the boreholes' locations. The topsoil thicknesses and type of existing fills and thicknesses may vary between the borehole locations, and these should be accounted in the estimation of the topsoil and existing fill volumes and excavation volumes.

Any required grade raising of the excavated subgrade to the design pavement subgrade level should consist of OPSS.MUNI 1010 Select Subgrade Material (SSM) or Granular B (Type I or II) material. The excavated inorganic (excluding topsoil, organics and debris) soils may also be reused for grade raise underneath pavements, but the inorganic soils should be carefully separated during excavation and approved as discussed in Section 6.7. Topsoil, organics, debris, wet soil, cobbles, and boulders should be separated from the excavated soils. Wet inorganic soils may be allowed to dry to an appropriate water content for placement and compaction. The separated inorganic soils should be inspected and approved by a qualified material consultant prior to use as backfill.

The grade raise fill should be placed in maximum loose lifts of 200 mm and compacted to at least 97% of SPMDD at ±2% of OMC. The granular engineered fills should extend laterally and be connected to the side drainage system to reduce local ponding of water inside the granular engineered fills.

As discussed in Section 6.10.2, frost compatibility must be maintained across the new utility trenches below the pavements by forming frost tapers. The trench backfill is also recommended to consist of granular engineered fill to reduce potential subgrade settlement along the trench line underneath the new pavements and concrete aprons or sidewalks.

The prepared subgrade should be inspected visually and proof-rolled by a qualified geotechnical consultant prior to the construction of the proposed pavements and concrete aprons or sidewalks. The prepared subgrade should also be protected from disturbance of construction traffic, freezing, excessive wetting, or drying. The asphalt or concrete structure subgrade should not be left open for a longer time to minimize the disturbance and excessive wetting, drying or freezing.

6.11.2 Recommended Pavement and Concrete Apron Structures

Detailed information of estimated traffic composition and vehicle counts have not been provided in the preparation of this report. Thus, preliminary pavement structures are recommended in this section assuming a 20-year ESAL of 225,000 for light-duty pavement and 1,000,000 for heavy-duty pavement. It is recommended that the detail traffic information is forwarded for our review and revision of the preliminary pavement structures as necessary.

The recommended structures for asphalt pavements and concrete aprons are provided in Table 8.

Portland cement concrete (PCC) rigid pavement was not considered for the proposed pavement areas.

Table 8: Recommended Structures for Asphalt Pavements and Concrete Aprons

Material		Structure Thicknesses (mm)		
		Light Duty(2)	Heavy Duty(3)	Concrete Apron(4)
Superpave Asphalt (1) OPSS.MUNI 1151	Surface Course - Superpave 12.5	40	40	-
	Binder Course - Superpave 19.0	40	60	-
Portland Cement Concrete	Portland Cement Concrete	-	-	200 (5)
Granular Material OPSS.MUNI 1010	Base Course - Granular A	150	150	150
	Subbase Course - Granular B, Type II	300	450	600
Subgrade	Prepared and Approved Subgrade as per Section 6.11.1. If silt or clay subgrade encountered, provide OPSS.MUNI 1860 Class II non-woven geotextile (Terrafix 360R or approved equivalent) over the prepared subgrade, as necessary, to prevent pumping of the subgrade into the Granular B Type II subbase.			

Notes:

(1) Asphalt cement should be Performance Graded Asphalt Cement (PGAC) 58-34 based on Zone 2 according to OPSS.MUNI 1101.

(2) Light-duty pavement is for the proposed parking lots with cars and light trucks only.

(3) Heavy-duty pavement is for heavy truck parking lots, drive lanes, access roads, delivery trucks, fire routes, and other equivalent areas (< 1,000,000 ESAL).

(4) Concrete apron is for loading docks, garbage pick-up locations, heavy vehicle stops/stations, and other equivalent areas.

(5) Provide a minimum of 100 mm thick rigid insulation underneath the slab as per Section 6.3.3.2. The selected rigid insulation should have the required compressive strength to resist the load from the slab or apron.

The concrete apron recommendation doesn't include the structural design of the slab. The concrete should be structurally designed for reinforcement, and concrete joint specifications and spacing in accordance with the applicable OPSD 551 and OPSD 552 series drawings. The Portland cement concrete classes of exposure should be determined based on Table 1 of CSA A23.1:19 based on the concrete slab application. The cement concrete should also satisfy the requirements in Table 2 to Table 4 of CSA A23.1:19. Degree of exposure for sulphate attack on concrete is discussed in Section 6.12.

As per OPSS.MUNI 501 (Subsection 501.08.02 Method A), the granular base and subbase materials should be uniformly compacted to 100% of SPMDD at ±2% of OMC.

As per OPSS.MUNI 310 (Table 10), Superpave 19.0, and Superpave 12.5 should be compacted to a minimum of 91% and 92%, respectively, of the Maximum Relative Density (MRD). Other applicable specifications in OPSS.MUNI 310 should also be followed in the placement of hot mix Superpave asphalts.

The material specifications, placement, and compaction should be inspected and approved by a qualified material consultant.

6.11.3 Drainage of Pavements and Concrete Aprons or Slabs

Effective drainage is key for long-term performance of asphalt pavements and concrete aprons or slabs. Positive drainage system should be provided to drain away surface runoff and subsurface water from the subgrade area through the shortest path possible. Ponding of water within the structures or underlying subgrade will weaken the materials and may lead to poor performance of the structures. Ponding of water will also promote frost heave actions which further deteriorate the structures.

The subgrade and surface of the structures should be crowned or sloped to promote drainage of water away from the structure surface, granular layers, and subgrade. Perforated pipe subdrains should be provided along the sides of the structures for the entire length. The subgrade and granular layers should be connected to the side subdrains to ensure effective drainage of all the surface and subsurface water. The perforated pipes should be enclosed with free drain granular material (OPSS.MUNI 1004 19 mm Clear Stone or approved equivalent) and wrapped up with Class II non-woven geotextile as per OPSS.MUNI 1860 (Terrafix 360R or approved equivalent). The subdrains should be connected to the catch basins. The subdrains should be installed in accordance with OPSS.MUNI 405.

Backfilling of catch basin laterals located below subgrade level should be completed using acceptable native soils or fill which matches the material types exposed on the lateral trench walls. This will reduce potential problems associated with differential frost heaving.

6.12 Corrosion and Cement Type

The basic chemical analyses results of two soil samples submitted to Eurofins Environment Testing are summarized in Section 5.4.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The sulphate results were compared with Table 3 of Canadian Standards Association (CSA A23.1:19) and indicated a **moderate to severe degree** of sulphate attack potential on concrete structures at this site. Accordingly, Type HS, HSb, HSLb, or HSe Portland cement can be considered for buried concrete substructures in contact with native soils, in accordance with Table 3 of CSA A23.1:19. Type HS cement shall not be used in reinforced concrete exposed to both chlorides and sulphates, including seawater (Table 3 of CSA A23.1:19). All imported soils should be tested for soluble sulphate contents. Tables 1 to 4 of CSA-A23.1-19 should be referenced for additional requirements and further information regarding concrete in contact with sulphates. In general, the properties of concrete in contact with soil or groundwater shall meet all the requirements of CSA A23.1:19.

The soil resistivity test results indicate a **highly corrosive** potential for corrosion of exposed ferrous metal at the site which should be considered in the design of substructures.

6.13 Special Provisions

Special Provisions should be included in the contract documents that notify contractors of the requirements for the following conditions, but not limited to, of the project construction:

- The native clay deposit may potentially be a sensitive Leda clay. The contractor should consider a suitable means and methods for excavation and construction of the foundations and other subsurface structures without causing disturbance to the potential sensitive Leda clay.
- Cobbles, boulders and rock fragments should be expected in the existing fill and native glacial till layers. The proposed excavation means and methods by the contractor should consider the removal requirement of these boulders and rock fragments.
- Variable competent bedrock surface topography should be expected at the project site. The contractor should consider the potential excavation of shallow bedrock surfaces or extension of foundations to deeper depths in case of deeper bedrock surface. The contractor’s means and method of excavation should also consider the bedrock excavation and diamond drilling or coring through bedrock for rock socketed pile construction.

- The contractor should consider a temporary dewatering system during the construction of shallow footings, raft slabs and/or rock socketed piles.

6.14 Additional Considerations

The soils at this site are sensitive to disturbance from ponded water, construction traffic, and frost.

All subgrade areas should be inspected by a qualified geotechnical consultant prior to backfilling to confirm that the correct/expected strata exist and that the bearing surfaces have been properly prepared. The placing and compaction of any engineered fill should be inspected to ensure that the materials used conform to the specifications from both grading and compaction requirements.

WSP should review the final drawings and specifications for this project prior to tendering to confirm that the recommendations in this report have been adequately interpreted.

Signature Page

WSP Canada Inc.

Arthur Kuitchoua Petke, P.Eng.
Project Manager, Geotechnical

Abraham Mineneh, PhD, P.Eng.
Senior Geotechnical Engineer

AKP/AM

https://wsponlinecan.sharepoint.com/sites/ca-ca0044112.4771/shared documents/06. deliverables/geotechnical report/full report/ca0044112.4771-draft rpt rev0 2025'02'14_geo 250 forestglade_final.docx

7.0 REFERENCES

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- CFEM. 2023. Canadian Foundation Engineering Manual, 5th edition. The Canadian Geotechnical Society, Canada. 888 pages
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- Mesri G (1989) A reevaluation of su(mob) = 0.22 σ'p using laboratory shear tests. Can Geotech J 26(1):162–164
- Jay Ameratunga, Nagaratman Sivakugan, Braja M.Das (2016). Correlations of soil and rock properties in geotechnical engineering, 236 pages
- CSA A23.1:19/CSA A23.2:19 (2019). Concrete materials and methods of concrete construction / Test methods and standard practices for concrete
- OPSD 3090.101 (2020). Foundation Frost Penetration Depths for Southern Ontario
- OPSS.MUNI 1010 (2013). Material Specifications for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material. 18 pages
- OPSS.MUNI 1151 (2018). Material Specifications for Superpave and Stone Mastic Asphalt Mixtures. 25 pages

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: WSP Canada Inc. (WSP) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to WSP by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. WSP cannot be responsible for use of this report, or portions thereof, unless WSP is requested to review and, if necessary, revise the report.

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The report is of a summary nature and is not intended to stand alone without reference to the instructions given to WSP by the Client, communications between WSP and the Client, and to any other reports prepared by WSP for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. WSP can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, WSP does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that WSP interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: WSP will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of WSP's report. WSP should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of WSP's report.

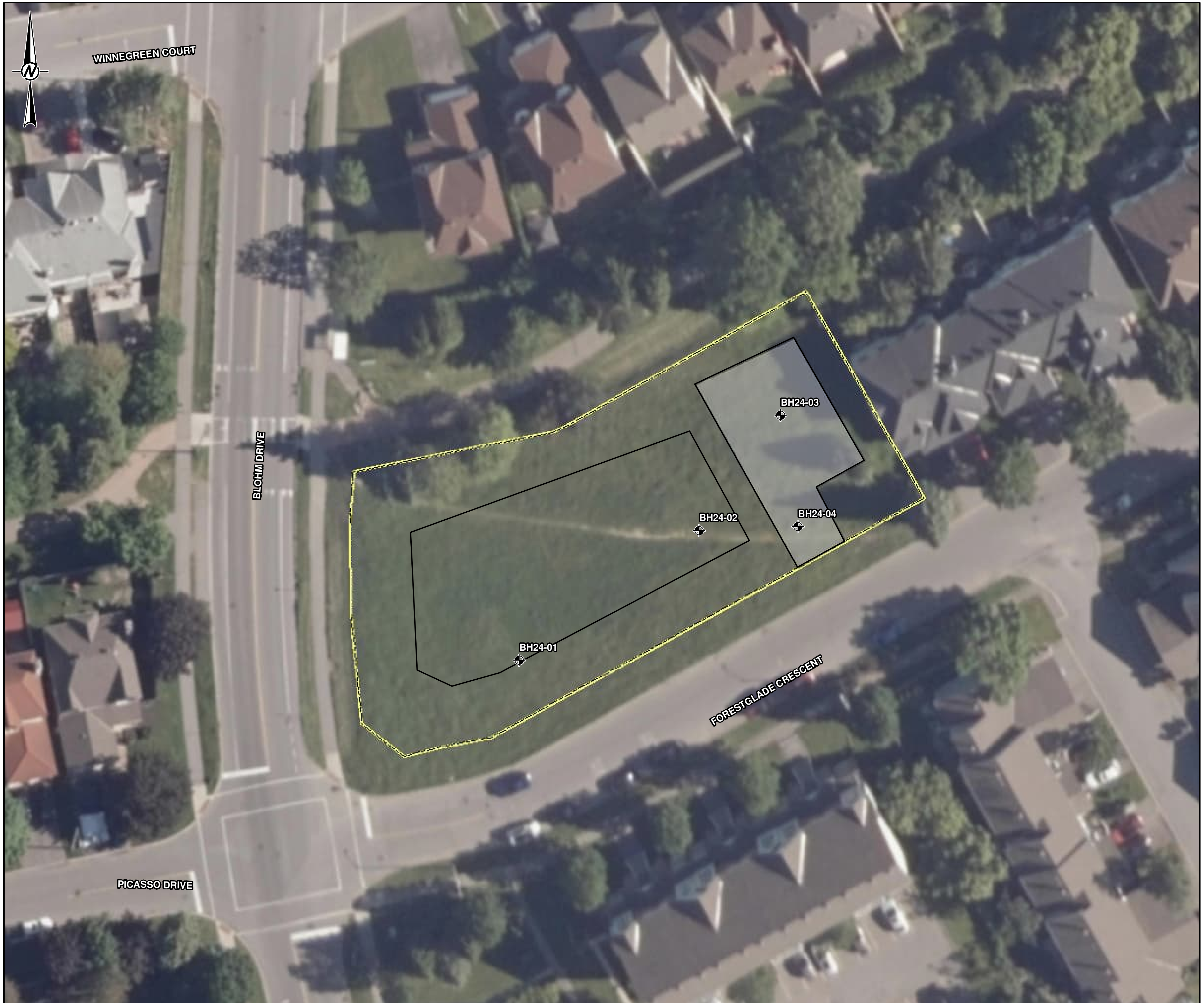
During construction, WSP should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of WSP's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in WSP's report. Adequate field review, observation and testing during construction are necessary for WSP to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, WSP's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that WSP be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that WSP be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

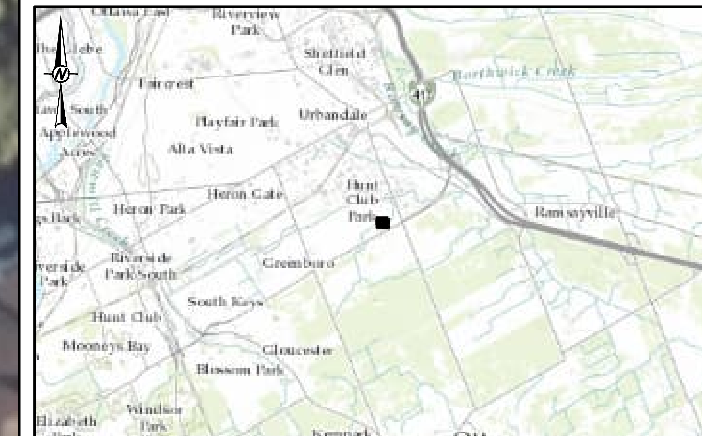
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. WSP takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

FIGURE

Figure 1 – Borehole Location Plan
Sketch 1 – Schematic Diagram for Perimeter Drain



KEY MAP



SCALE 1:150,000

LEGEND

- BOREHOLE LOCATION
- PROPOSED BUILDING
- PROPOSED PARKING LOT
- SITE BOUNDARY

0 5 10 20
1:500 METRES

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)
1. CONTAINS INFORMATION LICENSED UNDER THE OPEN GOVERNMENT LICENCE - ONTARIO
2. IMAGERY CREDITS: SOURCES: ESRI, HERE, GARMIN, INTERMAP, INCREMENT P CORP., GEBCO, USGS, FAO, NPS, NRCAN, GEOBASE, IGN, KADASTER NL, ORDNANCE SURVEY, ESRI JAPAN, METI, ESRI CHINA (HONG KONG), (C) OPENSTREETMAP CONTRIBUTORS, AND THE GIS USER COMMUNITY, OTTAWA, 2022
3. COORDINATE SYSTEM: NAD 1983 UTM ZONE 18N

CLIENT
OTTAWA ABORIGINAL COALITION

PROJECT
GEOTECHNICAL INVESTIGATION - PROPOSED BUILDING AND PARKING LOT DEVELOPMENT
250 FORESTGLADE CRESCENT, OTTAWA, ONTARIO

TITLE
BOREHOLE LOCATION PLAN

CONSULTANT

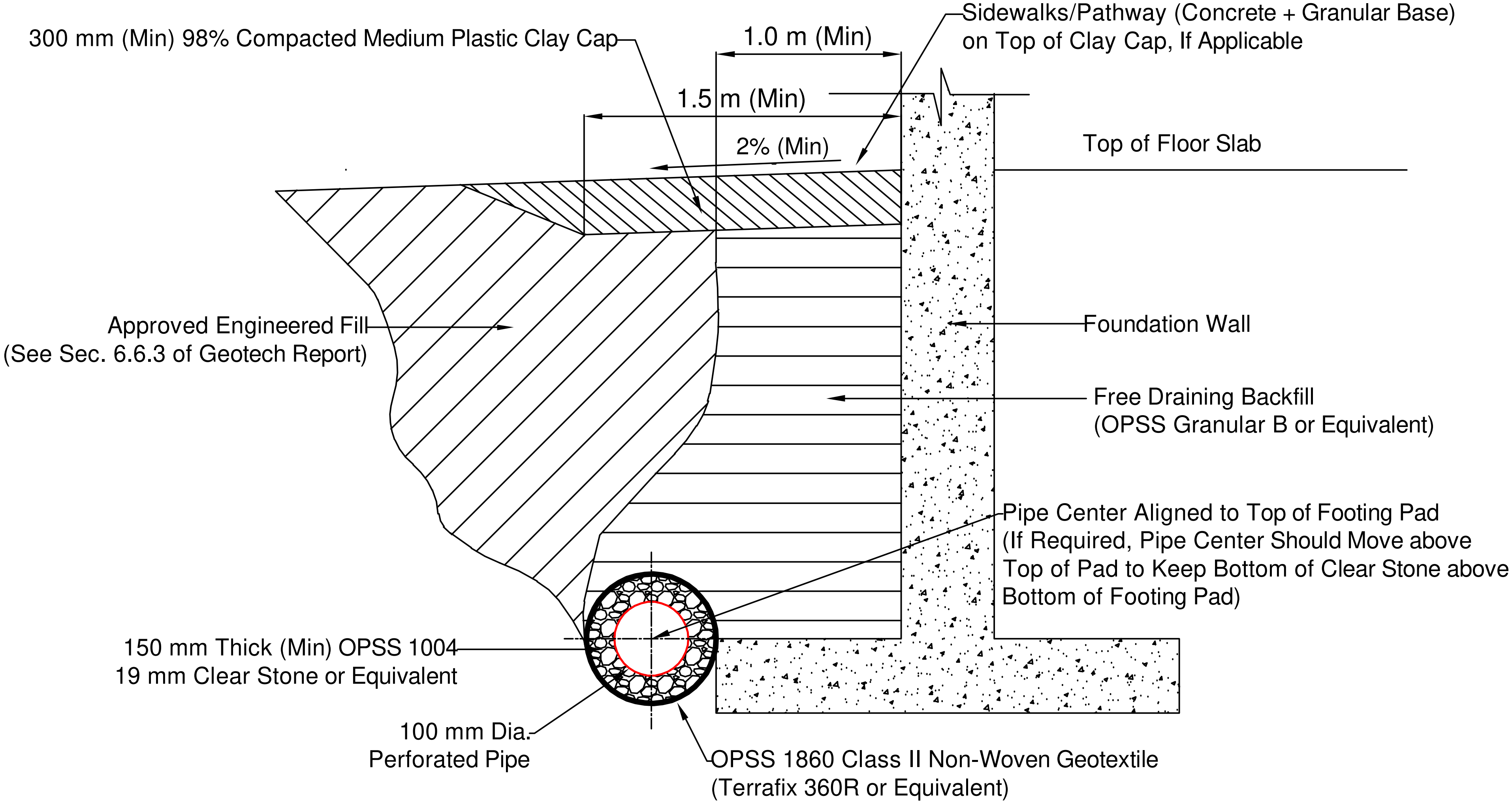
wsp

YYYY-MM-DD	2024-12-06
DESIGNED	----
PREPARED	CGE
REVIEWED	AKP
APPROVED	AM

PROJECT NO.	CONTROL	REV.	FIGURE
CA0044112.4771	0001	A	1

25mm
IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

SKETCH 1: SCHEMATIC DIAGRAM FOR PERIMETER DRAIN



NOTE: See additional details from Sec. 6.4, Sec. 6.8.3.4, and other sections of the geotechnical report.

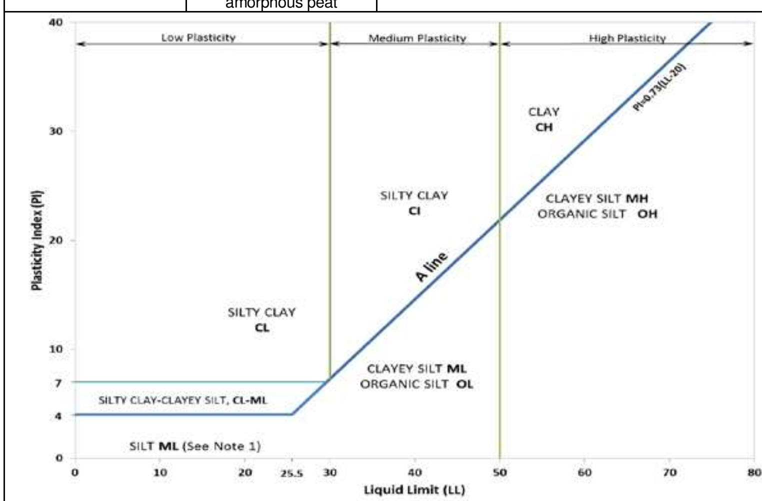
APPENDIX A

Record of Boreholes and Rock Drilling Sheets

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators: Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% (see Note 2)	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , w_L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, Gs)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
γ	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan\delta$
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan\phi'$
2 shear strength = (compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING CLASSIFICATION

Fresh (W1): no visible sign of rock material weathering.

Slightly Weathered (W2): discoloration indicates weathering of rock mass material on discontinuity surfaces. **Less than 5%** of rock mass is altered or weathered.

Moderately Weathered (W3): less than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Highly Weathered (W4): more than 50% of the rock mass is decomposed and/or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones.

Completely Weathered (W5): 100% of the rock mass is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

Residual Soil (W6): all rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid segments.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole, a discontinuity with a 90º angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

AXJ Axial Joint
BD Bedding
BC Broken Core
CC Continuous Core
CL Closed
CO Contact
CU Curved
CT Coated
FLT Fault
FOL Foliation
FR Fracture
GO Gouge
IN Infilled
IR Irregular
JN Joint
KV Karstic Void
K Slickensided
LC Lost Core
MB Mechanical Break
PL Planar
PO Polished
RO Rough
SA Slightly Altered
SH Shear
SM Smooth
SR Slightly Rough
SY Stylolite
UN Undulating
VN Vein
VR Very Rough

ISRM Intact Rock Material Strength Classification

Grade	Description	Approx. Range of Uniaxial Compressive Strength (MPa)
R0	Extremely weak rock	0.25 – 1.0
R1	Very weak rock	1.0 – 5.0
R2	Weak rock	5.0 – 25
R3	Medium strong rock	25 – 50
R4	Strong rock	50 -100
R5	Very strong rock	100 -250
R6	Extremely strong rock	>250

PROJECT: CA0044112.4771

LOCATION: N 5024630.06; E 452821.06

SPT/DCPT HAMMER: MASS, kg DROP, mm

RECORD OF BOREHOLE: BH24-1

BORING DATE: November 14, 2024

DRILL RIG: CME-55

SHEET 1 OF 1

DATUM: NAD 83

HAMMER TYPE:

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	83.19					GR SA SI CL
0	Power Auger; 200 mm Hollow Stem Auger	TOPSOIL (150 mm)	0.00					
		FILL - (ML) CLAYEY SILT, some sand, some gravel; brown, contains organics; w<PL, very stiff	0.15	1	SS	15		
1				2	SS	26		
		(CI) CLAY, medium plastic, trace gravel; brown; firm to stiff	81.67 1.52	3	SS	6	CHEM	
2				4	SS	6		
3				5	SS	14		
4		- spoon refusal at 4.06 m, rock fragments	79.13 4.06	6	SS	50/0		
		END OF BOREHOLE For bedrock coring details refer to Record of Drillhole BH24-01.						

Chart column headings: DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80); SHEAR STRENGTH Cu, kPa (nat V. + Q - ●; rem V. ⊕ U - ○) (20, 40, 60, 80); HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6}, 10^{-5}, 10^{-4}, 10^{-3}); WATER CONTENT PERCENT (Wp —○W— Wl) (10, 20, 30, 40); GRAIN SIZE DISTRIBUTION (%)

DEPTH SCALE

1 : 50

LOGGED: AKP

CHECKED: OB

PROJECT: CA0044112.4771

RECORD OF DRILLHOLE: BH24-1

SHEET 1 OF 1

LOCATION: N 5024630.1 ;E 452821.1

DRILLING DATE: November 14, 2024

DATUM: NAD 83

DRILL RIG: CME-55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: George Downing Estate Drilling

NOTE:
For abbreviations, symbols and descriptions refer to
LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY TOTAL CORE % (80 60 40 20)	RECOVERY SOLID CORE % (80 60 40 20)	R.Q.D. % (80 60 40 20)	FRACT. INDEX PER (5 10 15 20)	DIP w.r.t. CORE AXIS (0 30 60 90)	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3})	WEATHERING INDEX (W1 W2 W3 W4 W5 W6)	FEATURES
		Continued from Record of Borehole BH24-1															
	Rotary Drill, NQ Coring	Moderately to slightly weathered with clay infill, bedded, grey, fine grained, non-porous, SHALE - Excellent to fair quality rock		79.13 4.06	1												
5					2												
				77.45													
		END OF DRILLHOLE		5.74													
6																	
7																	
8																	
9																	
10																	
11																	
12																	
13																	
14																	

DEPTH SCALE

1 : 50

LOGGED: AKP

CHECKED: AKP

PROJECT: CA0044112.4771

LOCATION: N 5024646.76; E 452844.26

SPT/DCPT HAMMER: MASS, kg DROP, mm

RECORD OF BOREHOLE: BH24-2

BORING DATE: November 14, 2024

DRILL RIG: CME-55

SHEET 1 OF 1

DATUM: NAD 83

HAMMER TYPE:

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT / NOTES	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION / GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0	Power Auger 200 mm Hollow Stem Auger	GROUND SURFACE	83.48						
		TOPSOIL (150 mm)	0.00						
		FILL - (SP-GP) gravelly SAND, some silt; dark brown, contains organics; moist, compact	0.15 / 83.08	1A	SS	12			Hole Plug
		(CH) CLAY high plastic with sand, trace gravel; grey brown to grey; stiff to firm	0.40	1B					
1				2	SS	15			
				3	SS	12		CHEM	
2		- thin sand seam		4	SS	11			12 23 23 42
		- firm		5	SS	4	58 / 53		Sand
3				6	SS	3			Screen
		- soft							
4		(SP-GP) SAND and GRAVEL, contains rock fragments; grey (TILL); moist, compact to very dense	79.72 / 3.76	7	SS	26			December 02, 2024
		- spoon refusal	79.09 / 4.39	8	SS	50/ 0.15			
		END OF BOREHOLE For bedrock coring details refer to Record of Drillhole BH24-02.							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp —○W— Wl : 10 20 30 40

DEPTH SCALE

1 : 50

LOGGED: AKP

CHECKED: OB

PROJECT: CA0044112.4771

RECORD OF DRILLHOLE: BH24-2

SHEET 1 OF 1

LOCATION: N 5024646.8 ;E 452844.3

DRILLING DATE: November 14, 2024

DRILL RIG: CME-55

DRILLING CONTRACTOR: George Downing Estate Drilling

DATUM: NAD 83

INCLINATION: -90° AZIMUTH: ---

NOTE: For abbreviations, symbols and descriptions refer to **LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY**

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FEATURES
		Continued from Record of Borehole BH24-2	79.09		
	Rotary Drill / NQ Coring	Highly weathered bedrock	4.39	1A	Hole Plug
5	Rotary Drill / NQ Coring	Moderately to slightly weathered, bedded, grey, fine grained, non-porous to slightly porous, SHALE -Poor quality rock	78.53 4.95	1B	Hole Plug
		END OF DRILLHOLE	77.74 5.74		

Column headings: SYMBOLIC LOG; FLUSH RETURN; RECOVERY (TOTAL CORE % 80 60 40 20; SOLID CORE % 80 60 40 20); R.Q.D. % 80 60 40 20; FRACT. INDEX PER 5 10 15 20; DISCONTINUITY DATA (DIP w.r.t. CORE AXIS 0 30 60 90; TYPE AND SURFACE DESCRIPTION; Jr; Ja); HYDRAULIC CONDUCTIVITY K, cm/sec 10^{-6} 10^{-5} 10^{-4} 10^{-3}; WEATHERING INDEX W1 W2 W3 W4 W5 W6

Depth scale: 5, 6, 7, 8, 9, 10, 11, 12, 13, 14

DEPTH SCALE

1 : 50

LOGGED: AKP

CHECKED: AKP

PROJECT: CA0044112.4771

LOCATION: N 5024661.60; E 452854.74

SPT/DCPT HAMMER: MASS, kg DROP, mm

RECORD OF BOREHOLE: BH24-3

BORING DATE: November 14, 2024

DRILL RIG: CME-55

SHEET 1 OF 1

DATUM: NAD 83

HAMMER TYPE:

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT (W)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION / GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE	83.65						
0	Power Auger, 200 mm Hollow Stem Auger	TOPSOIL (130 mm)	0.00						
		FILL - (ML) CLAYEY SILT, some gravel, trace sand with rock fragments and boulders; light brown; stiff	0.13	1	SS	12			
1		(SM) SILTY SAND with gravel (Potential Fill); grey-brown with rock fragments and boulders; moist, dense to compact	83.04 / 0.61	2	SS	32	~9		26 46 (28)
				3	SS	19	~8		
2		END OF BOREHOLE	81.82 / 1.83						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ ; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp —○W— Wl ; 10 20 30 40

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

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DEPTH SCALE

1 : 50

wsp

LOGGED: AKP

CHECKED: OB

RECORD OF BOREHOLE: BH24-4

PROJECT: CA0044112.4771
LOCATION: N 5024647.29; E 452856.97
SPT/DCPT HAMMER: MASS, kg DROP, mm

BORING DATE: November 14, 2024
DRILL RIG: CME-55

SHEET 1 OF 1
DATUM: NAD 83
HAMMER TYPE:

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80) / SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}) / WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION / GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		GROUND SURFACE		83.56							
0	Power Auger, 200 mm Hollow Stem Auger	TOPSOIL (180 mm)		0.00							
		FILL - (SP-GP) gravelly SAND, some silt, trace clay; brown with rock fragments; moist, compact		83.38 / 0.18	1	SS	20		○		
1		(SM) SILTY SAND with gravel, contains rock fragments (Potential Fill); light brown to grey, contains cobbles; moist, compact - boulder/rock fragments		82.65 / 0.91	2	SS	10		○		
					3	SS	29				25 42 (33)
		- boulder/rock fragments									
2		END OF BOREHOLE		81.73 / 1.83							

Depth scale: 0 1 2 3 4 5 6 7 8 9 10

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DEPTH SCALE
1 : 50

LOGGED: AKP
CHECKED: OB

APPENDIX B

Record of Rock Core Photos

24-01 (Dry)

Core Box 1 of 1

Elevation 79.13 m Top of

Elevation 77.45 m
End of Borehole

wsp	**Geotechnical Investigation** **Proposod Buildind and Parking Lot** **250 Forestglade Crescent** **Ottawa (ON)**	Project No. CA0044112.4771 Drawn: AKP Date: 2024-12-20 Checked: AKP Review: AM	**Figure B1**

24-01 (Wet)

Core Box 1 of 1

Elevation 79.13 m Top of Bedrock

Elevation 77.45 m End of Borehole

wsp	**Geotechnical Investigation** **Proposod Buildind and Parking Lot** **250 Forestglade Crescent** **Ottawa (ON)**	Project No. CA0044112.4771 Drawn: AKP Date: 2024-12-20 Checked: AKP Review: AM	**Figure B2**

24-02 (Dry)
Core Box 1 of 1

Elevation 79.09 m Top of Bedrock

Elevation 77.74 m
End of Borehole

wsp	**Geotechnical Investigation** **Proposod Buildind and Parking Lot** **250 Forestglade Crescent** **Ottawa (ON)**	Project No. CA0044112.4771 Drawn: AKP Date: 2024-12-20 Checked: AKP Review: AM	**Figure B3**

24-02 (Wet)

Core Box 1 of 1

Elevation 79.09 m Top of Bedrock

Elevation 77.74 m
End of Borehole

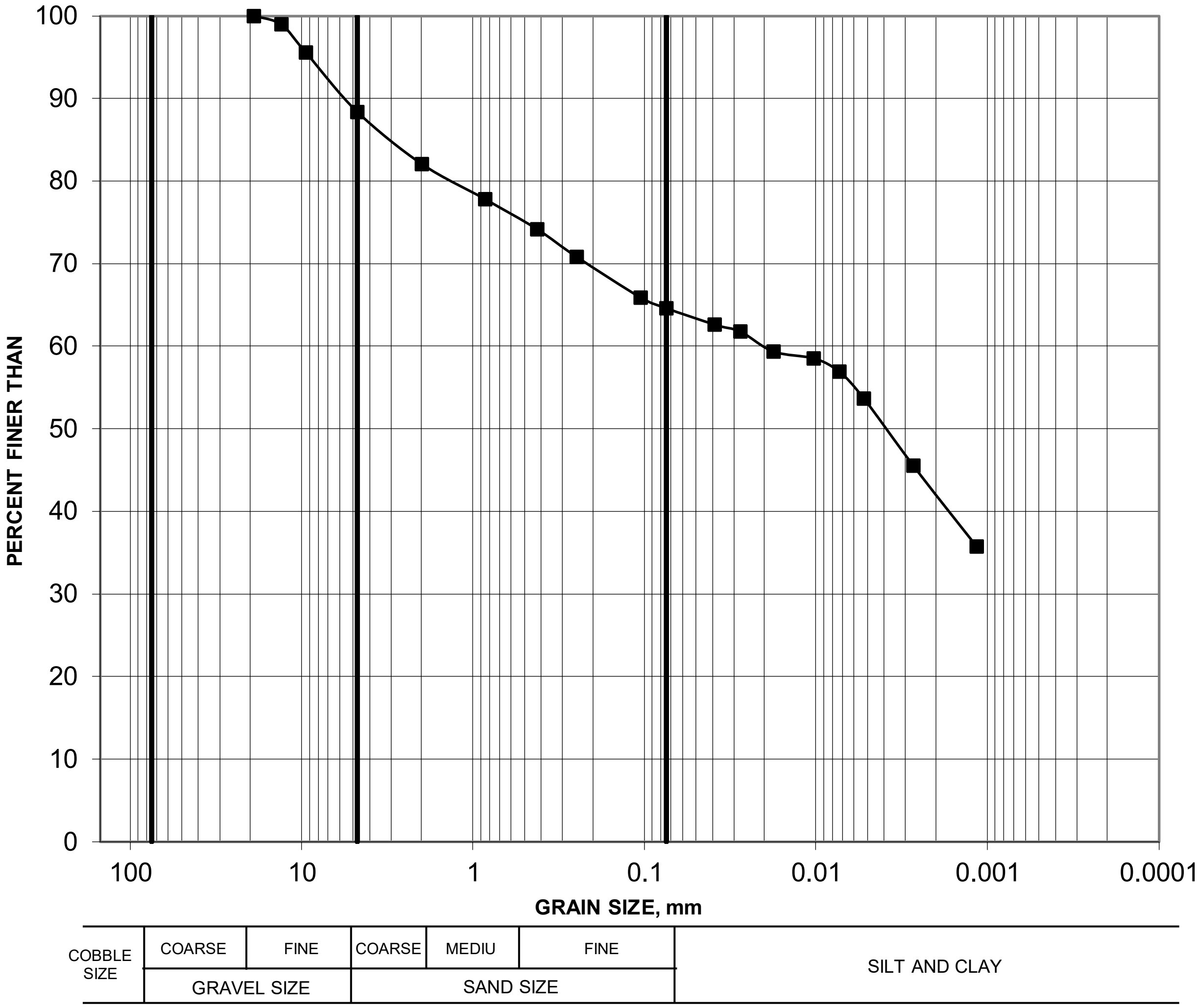
wsp	**Geotechnical Investigation** **Proposod Buildind and Parking Lot** **250 Forestglade Crescent** **Ottawa (ON)**	Project No. CA0044112.4771 Drawn: AKP Date: 2024-12-20 Checked: AKP Review: AM	**Figure B4**

APPENDIX C

Geotechnical Laboratory Testing Results

GRAIN SIZE DISTRIBUTION	FIGURE 2

HIGH PLASTIC CLAY WITH SAND, TRAVE GRAVEL



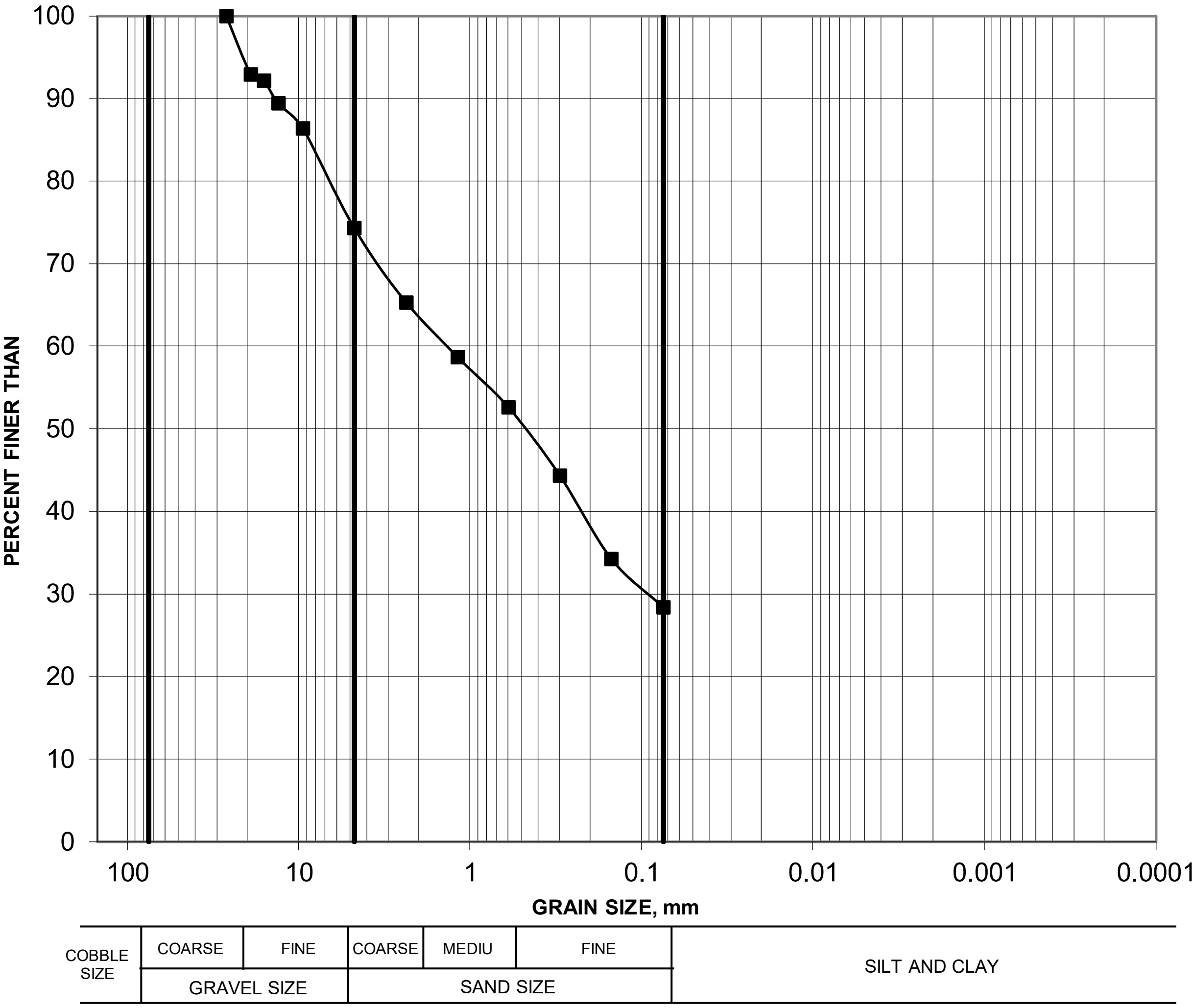
				Constituents (%)			
Borehole		Sample	Depth (m)	Gravel	Sand	Silt	Clay
■	24-02	4	1.83-2.44	12	23	23	42

Project: CA0044112.4771

Created by: CW
Checked by: MI

GRAIN SIZE DISTRIBUTION	FIGURE 3

SILTY SAND WITH GRAVEL

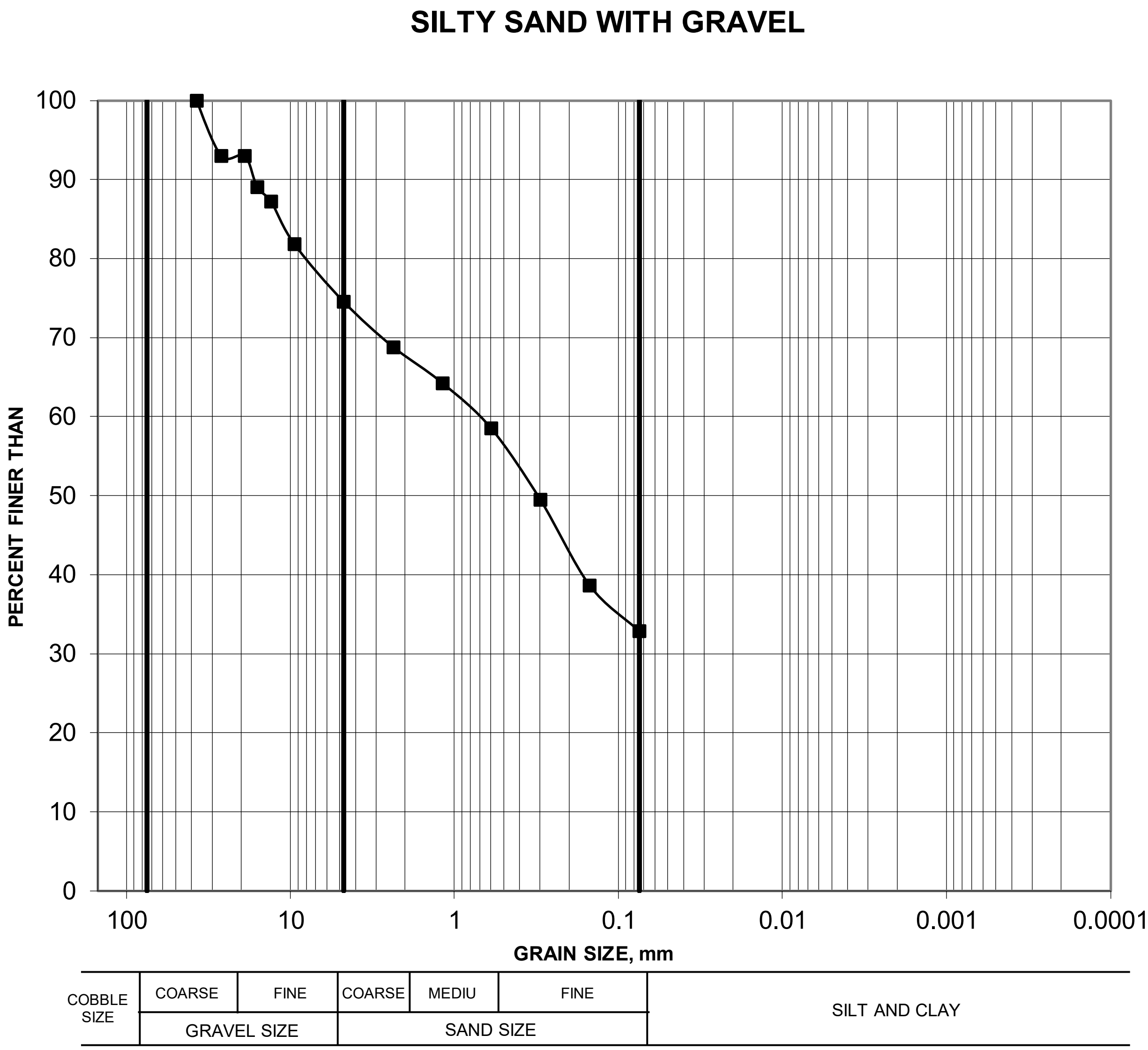


			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
24-03	2	0.61-1.22	26	46	28	

Project: CA0044112.4771

Created by: CW
Checked by: MI

GRAIN SIZE DISTRIBUTION	FIGURE 4

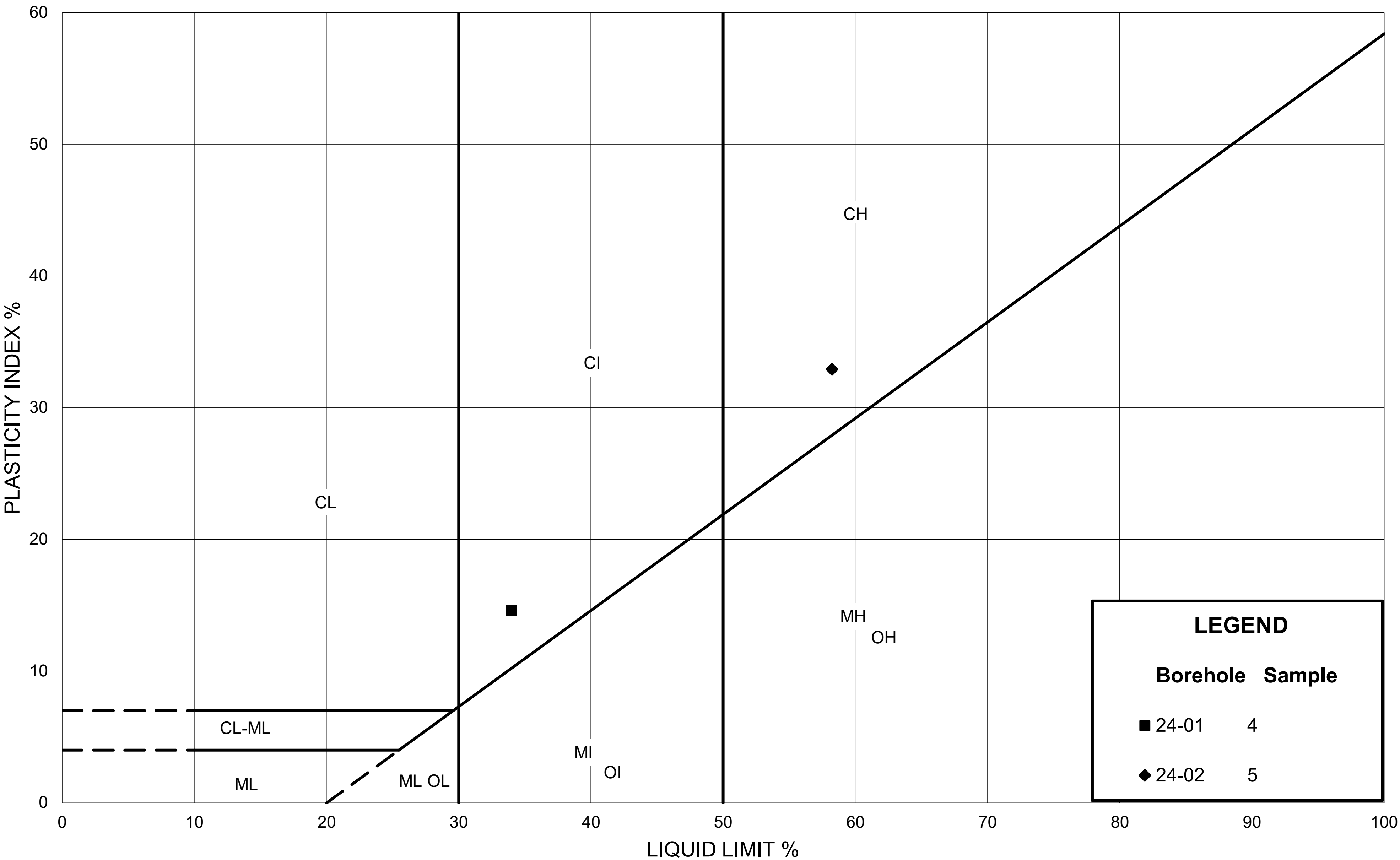


			Constituents (%)			
Borehole	Sample	Depth (m)	Gravel	Sand	Silt	Clay
24-04	3	1.22-1.83	25	42	33	

Project: CA0044112.4771

Created by: CW

Checked by: MI



PLASTICITY CHART

Figure:	5
Project:	CA0044112.4771
Created By:	CW Checked By: MI

APPENDIX D

Results of Basic Chemical Analyses

Certificate of Analysis

Client:	WSP Canada Inc.(Ottawa) 1931 Robertson Road Ottawa, Ontario K2H 5B7	Report Number:	3012641
Attention:	Mr. Arthur Kuitchoua Petke	Date Submitted:	2024-11-18
PO#:		Date Reported:	2024-11-26
Invoice to:	WSP Canada Inc.	Project:	CA0044112.4771/1300
		COC #:	917976

Dear Arthur Kuitchoua Petke:

Please find attached the analytical results for your samples. If you have any questions regarding this report, please do not hesitate to call (613-727-5692).

Report Comments:

Patrick Jacques 2024.11.26 14:30:27 -05'00'

APPROVAL: ______________________

Patrick Jacques, Chemist

All analysis is completed at Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) unless otherwise indicated.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by CALA, Canadian Association for Laboratory Accreditation to ISO/IEC 17025 for tests which appear on the scope of accreditation. The scope is available at: https://directory.cala.ca/.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is licensed by the Ontario Ministry of the Environment, Conservation, and Parks (MECP) for specific tests in drinking water (license #2318). A copy of the license is available upon request.

Eurofins Environment Testing Canada Inc. (Ottawa, Ontario) is accredited by the Ontario Ministry of Agriculture, Food, and Rural Affairs for specific tests in agricultural soils.

Please note: Field data, where presented on the report, has been provided by the client and is presented for informational purposes only. Guideline values listed on this report are provided for ease of use (informational purposes) only. Eurofins recommends consulting the official provincial or federal guideline as required. Unless otherwise stated, measurement uncertainty is not taken into account when determining guideline or regulatory exceedances.

Certificate of Analysis

eurofins | Environment Testing

Client:	WSP Canada Inc.(Ottawa) 1931 Robertson Road Ottawa, Ontario K2H 5B7	Report Number:	3012641
Attention:	Mr. Arthur Kuitchoua Petke	Date Submitted:	2024-11-18
PO#:		Date Reported:	2024-11-26
Invoice to:	WSP Canada Inc.	Project:	CA0044112.4771/1300
		COC #:	917976

				Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1751501 Soil 2024-11-14 24-02 Sa3/4-6'	1751502 Soil 2024-11-14 24-01 Sa3/5-7'
Group	**Analyte**	**MRL**	**Units**	**Guideline**		
Anions	Cl	0.002	%		0.003	<0.002
	SO4	0.01	%		0.11	0.23
General Chemistry	Electrical Conductivity	0.05	mS/cm		0.91	1.08
	pH	2.00			7.67	7.55
	Resistivity	1	ohm-cm		1099	926

Guideline = *** = Guideline Exceedence**

Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

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Client:	WSP Canada Inc.(Ottawa) 1931 Robertson Road Ottawa, Ontario K2H 5B7	Report Number:	3012641
Attention:	Mr. Arthur Kuitchoua Petke	Date Submitted:	2024-11-18
PO#:		Date Reported:	2024-11-26
Invoice to:	WSP Canada Inc.	Project:	CA0044112.4771/1300
		COC #:	917976

QC Summary

Analyte	Blank	QC % Rec	QC Limits
Run No 469150 **Analysis/Extraction Date** 2024-11-25 **Analyst** M B **Method** AG SOIL			
SO4	<0.01 %	100	70-130
Run No 469157 **Analysis/Extraction Date** 2024-11-25 **Analyst** AsA **Method** C CSA A23.2-4B			
Chloride	<0.002 %	91	75-125
Run No 469163 **Analysis/Extraction Date** 2024-11-25 **Analyst** IP **Method** Cond-Soil			
Electrical Conductivity	<0.05 mS/cm	100	90-110
pH	6.30	100	90-110
Resistivity			

Guideline = *** = Guideline Exceedence**

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