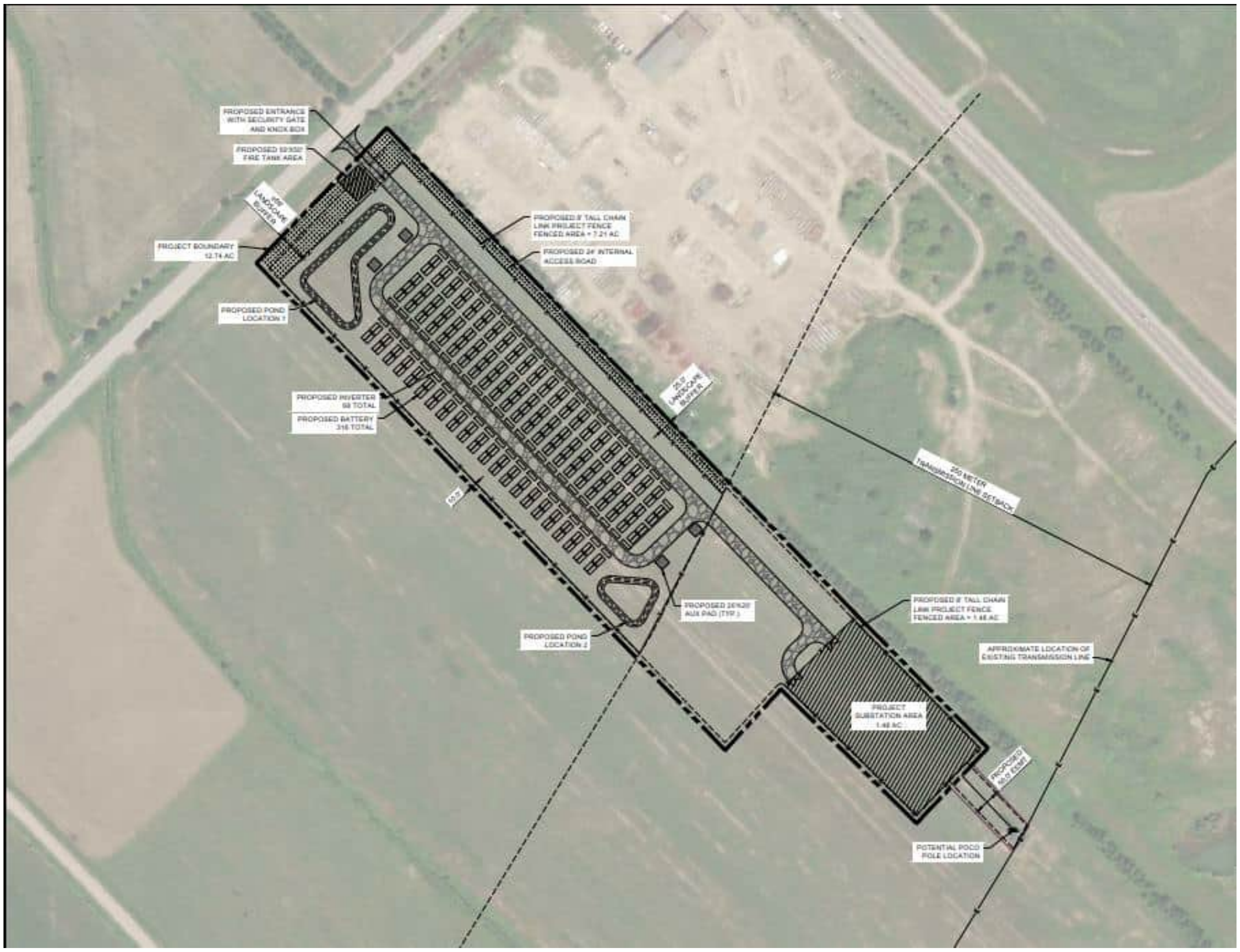
ENGINEERING CONSULTING SERVICES
GEOTECHNICAL INVESTIGATION REPORT
6235 GUELPH ROAD,CENTRE WELLINGTON, ON.



Project No.: CCO-242294-02

Prepared for:

Aypa Power
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Site Address:

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Centre Wellington, ON.

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November 15, 2023

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Re: Geotechnical Investigation Report
6234 Guelph Road, Centre Wellington, ON.

1.0 INTRODUCTION

McIntosh Perry Consulting Engineers Ltd. (MPCE) was retained by Aypa Power (Client) to carry out a geotechnical investigationfor the proposed location of the Battery Energy Storage System (BESS) at the property located at 6234 Guelph Road, Centre Wellington, Ontario. ("Site" – Figure -1).

The property consists of an existing vegetated farm. The proposed platform is located on the north half of the property (Site) and will comprise of a platform for the proposed batteries over at the elevation of the existing subgrade.

The purpose of the geotechnical investigation was to determine the subsurface conditions and groundwater observations at the site by means of boreholes, field, and laboratory tests.

Based on the information obtained, the geotechnical characteristics of the subsurface soils were estimated, and site conditions described, to provide geotechnical recommendations for the foundation elements of the above-mentioned structures:

- Structure foundations
- Permanent drainage
- Excavation support and lateral earth pressures
- Excavation and backfill for buried services and utilities.

The fieldwork and geotechnical analyses of this investigation were completed in general accordance with the geotechnical elements described in the MPCE Quote No. PCO-242294-00.

Figure-1 Site Location

2.0 FIELD AND LABORATORY INVESTIGATIONS

2.1 FIELD INVESTIGATION

MPCE visited the site before the drilling investigation to mark the proposed borehole locations in order to obtain utilities clearance to identify the location of underground infrastructures. Public and private utility authorities were informed, and all utility clearance documents were obtained before the commencement of the drilling work. The drilling work was carried out between October 25 to 26, 2023.

Field investigation included drilling four boreholes to a depth ranging between 7.7 m and 7.9 m below the existing ground surface (El. 411.9 to 416.1 meter above sea level masl) were carried out. The location of the boreholes is shown on Figure -2. The equipment used for drilling was owned and operated by Drilltech Drilling Ltd. of Newmarket, Ontario. Boreholes BH-1 and BH-4 were advanced and equipped with solid stem auger MARL M5T track-mounted drill rig. One monitoring well was installed on borehole BH-1 to monitor the groundwater level at the site.

A list of borehole depths and elevation is given in Table 2-1. Field tests were carried out during drilling to determine the engineering parameters of the soil, In-situ tests included Standard Penetration Tests (SPT) along with Pocket

Penetrometer Tests were conducted during the field exploration. Water level observations were made in each borehole upon its completion.

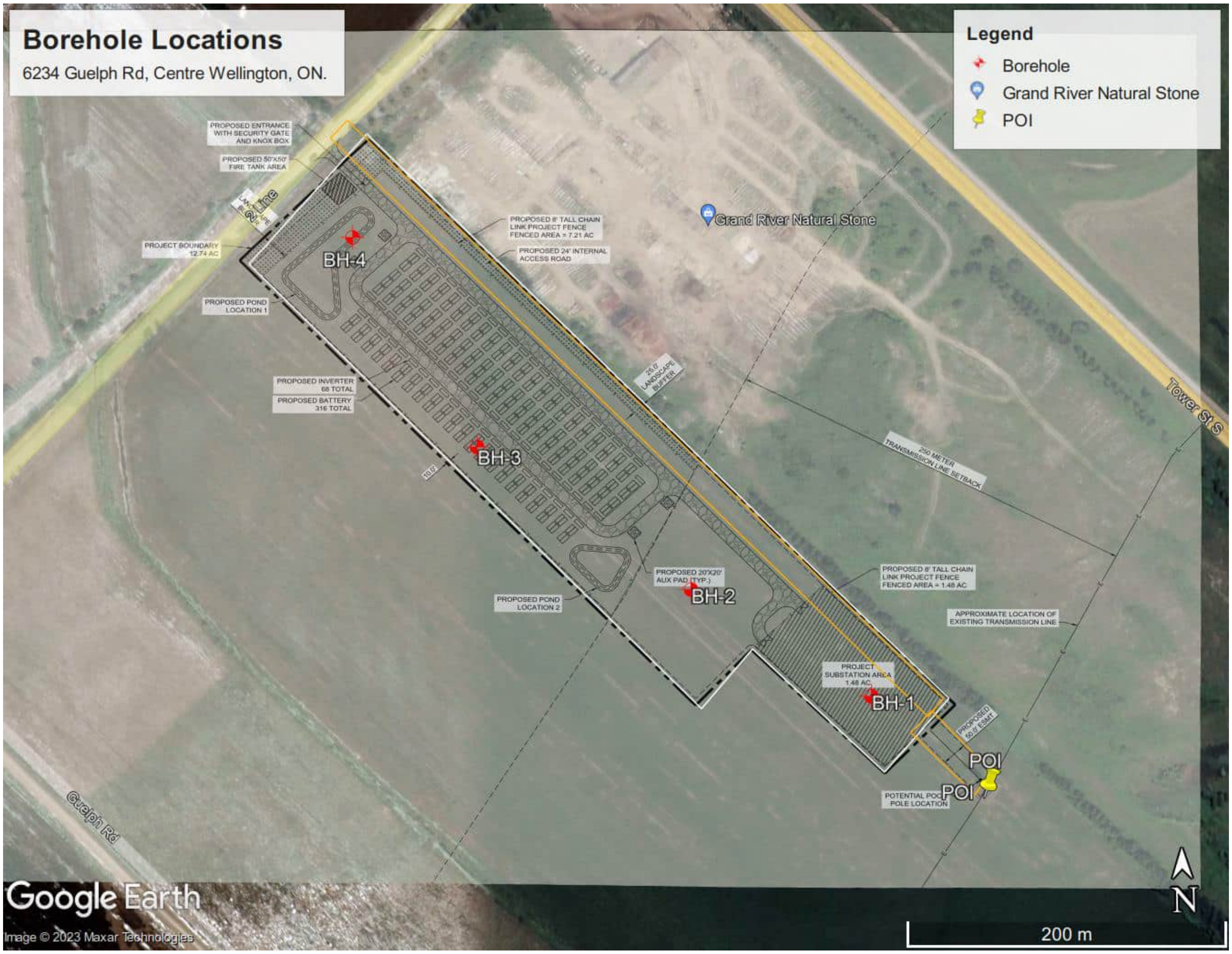


Figure-2 Borehole Locations Plan

Table 2-1- Borehole Designation, Locations, and Depth

BH No.	Coordinates (UTM 17T)		Surface El.	Total Borehole Depth (m)	Monitoring Well Depth (m)
	Northing (m)	Easting (m)	(m asl)		
BH-1	4837035.372	551940.193	412.4	7.9	7.6
BH-2	4837103.147	551828.657	416.1	7.7	N/A
BH-3	4837194.639	551695.406	415.0	7.8	N/A
BH-4	4837328.77	551617.676	411.9	7.7	N/A

The subsurface strata were sampled at regular intervals of depth using a split-spoon sampler, following the procedure as detailed in the ASTM-D1586 for the Standard Penetration Test. SPT' N' blow counts were recorded as per the subject ASTM procedure to indicate the compactness condition or the consistency of the sampled soil material. The SPT 'N' values thus obtained are indicated on the Record of Borehole Sheets (Refer to Appendix A).

The fieldwork was carried out under the full-time supervision of a MPCE field engineer who directed the exploration and sampling operation, logged borehole data, and took custody of the soil samples retrieved for subsequent laboratory testing and identification. Table 2-1 presents a summary of the borehole details of the MPCE geotechnical investigation program. Approximate locations of the drilled boreholes are shown in Figure-1.

2.2 Laboratory Investigations

All soil samples were taken to our CCIL accredited Toronto laboratory for final visual assessment, classification, and testing. All samples were visually examined and classified in general accordance with the Unified Soil Classification System, and Canadian Foundation Engineering Manual (CFEM).

A routine laboratory testing program consisting of natural water content, grain size distribution analysis, hydrometer analysis and Atterberg Limits were carried out on selected representative soil samples retrieved from the SPT split-spoon sampler.

Geotechnical laboratory test procedures are listed below.

- ASTM D2216 – Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (LS-701)
- ASTM D6913 - Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis
- ASTM D7928 – Standard Test Method for Particle-Size Distribution (Gradation) of Fine-Grained Soils Using the Sedimentation (Hydrometer) Analysis

The soil samples recovered during the investigation will be stored in our laboratory for a period of 30 days following the submission of the final geotechnical report, after which they will be discarded unless further instructions are received.

3.0 SUBSURFACE CONDITIONS

3.1 OVERVIEW

The subsurface conditions encountered in the boreholes are shown on the borehole records provided in Appendix A. The boreholes records include soil stratification at the borehole locations with detailed soil descriptions and selected physical properties for each stratum encountered.

Variations in the soil stratification may occur and should be expected between the borehole's locations and elsewhere on the site. Furthermore, since the internal diameter of the spoon used in the Standard Penetration Test (SPT) is 38 mm which is less than the maximum size of the gravel (75mm), the grain size test results and soil classification may not reflect the entire gravel size fraction.

The sub-surface consists of the following eight stratigraphic layers:

- Topsoil Layer
- Fill Soil layer,
- Native Cohesionless Deposits

Unless otherwise mentioned, all SPT 'N' results quoted are for SPT spoon penetrations of 300mm as per the subject ASTM. Supplementary information supporting the above overall subsurface observations, where available and indicated below. However, it should be borne in mind the below descriptions are based on and limited to, some generalizations of the actually verified soil information intercepted in the boreholes and documented in the borehole logs.

3.2 TOPSOIL LAYER

Topsoil Layer was covering all the boreholes, the thickness of the topsoil layer ranged between ranged between 0.2 to 0.5m.

3.3 FILL SOIL LAYER

Fill Soil layer was intercepted at all boreholes and generally comprised of sandy silt. The Fill Soil depth, thickness, composition is given in Table 3-1.

Table 3-1: Location, Thickness, and Composition of the Fill Soil Layer

BH No.	Depth (mbgs)	Top Elevation (masl)	Bottom Elevation (masl)	Thickness (m)	Description
BH-1	0.4	412.1	411.5	0.6	sandy silt, some clay, trace of gravel & topsoil
BH-2	0.2	415.8	413.9	1.9	sandy silt, some clay, trace of gravel & topsoil
BH-3	0.3	414.7	414.1	0.6	silty sand, some clay, trace of gravel, roots & topsoil.

BH-4	0.5	411.4	409.2	2.2	sandy silt, some clay, trace of gravel & topsoil

Grain size analysis test result of one representative sample from the Fill Soil Layer is shown in Table 3-2, and the corresponding graphical plot is shown in, Appendix B.

Table 3-2 - Grain Size Distribution Summary – Fill Soil Layer

BH No./ SS No.	Size Fraction (%)					Remarks
	Gravel	Sand	Silt	Clay	Fines	Appendix B
BH-05 / SS2	10	35	43	12	-	Moisture Content (%) = 11

The Standard Penetration Test (SPT) 'N' values widely ranged from 3 to 22 blows per 300mm penetration, with an average value of 11 indicating loose to compact compactness.

3.4 Native Cohesionless Deposits

The native soil consisted of a cohesionless deposits dominantly comprised of Sandy Silt Till. An interbedded layer of Sand and Silt was intercepted within the Sandy Silt Till in Borehole BH-1 and overlies the Sandy Silt Layer at BH-3.

The native cohesionless deposit layer depth, thickness, composition is given in Table 3-3.

Table 3-3: Location, Thickness, and Composition of the Native Cohesionless Deposit Layer

BH No.	Depth of the Layer Below the Ground Surface (mbgs)	Top Elevation (masl)	Bottom Elevation (masl)	Thickness (m)	Description
BH-1	0.9	411.5	408.1	3.4	Sandy Silt Till, some clay, trace of gravel
BH-1	4.3	408.1	405.2	2.9	Sand and Silt Till, trace of clay and gravel
BH-1	7.2	405.2	404.5*	N/A	Sandy Silt Till, trace of clay and gravel
BH-2	2.1	413.9	408.4*	N/A	Sandy Silt Till, trace of clay and gravel
BH-3	0.9	414.1	412.0	2.1	Sand and Silt Till, trace of clay and gravel
BH-3	3.0	412.0	408.9	3.1	Sandy Silt Till, trace of clay and gravel

BH-3	6.1	408.9	407.2*	N/A	Silt Till, trace to some sand and clay, trace gravel
BH-4	2.7	409.2	404.4*	N/A	Sandy Silt Till, trace of clay and gravel

*Termination Depth

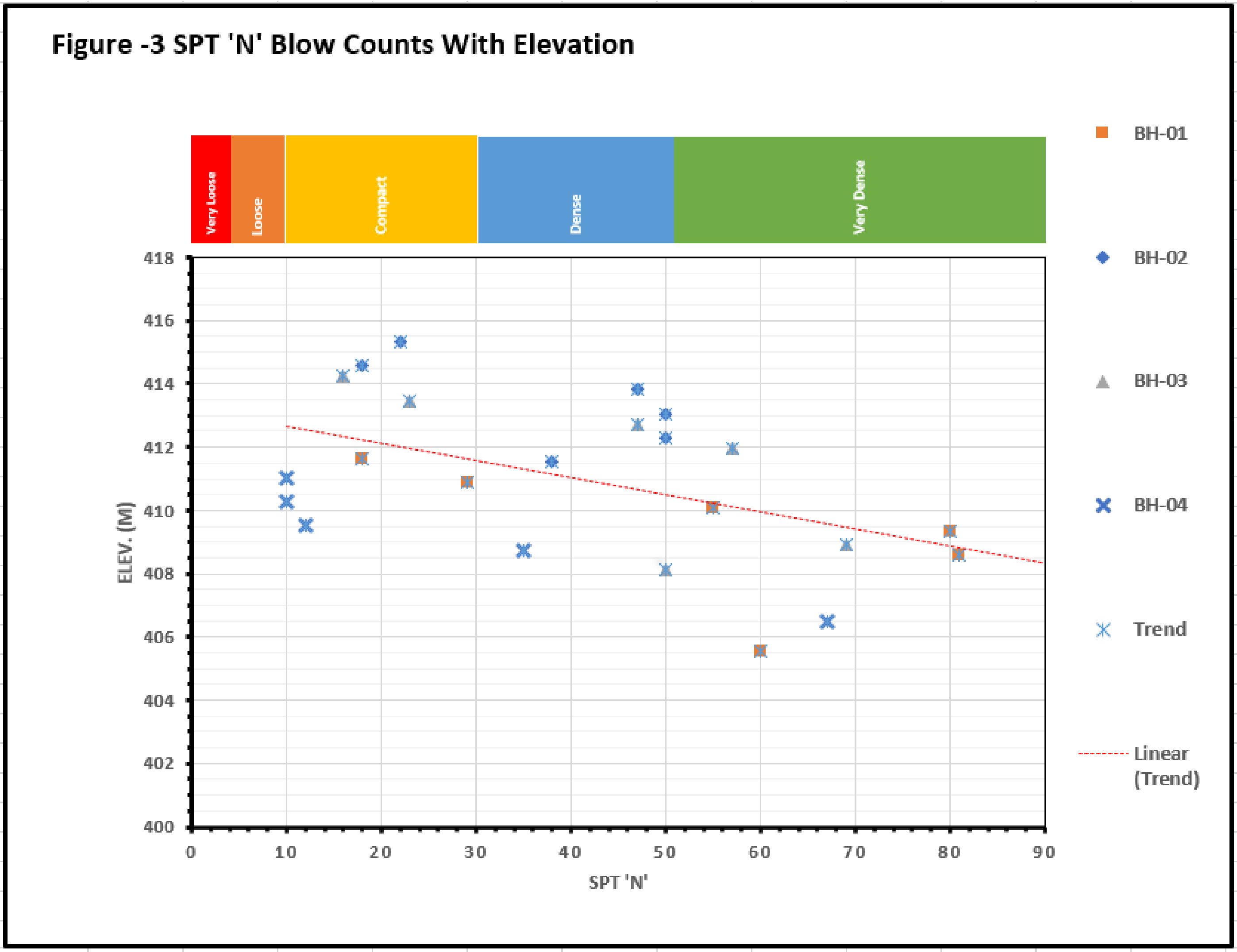
Grain size analysis test results of one representative samples from the Native Cohesionless Deposit Layer are shown in Table 3-4, and the corresponding graphical plot is shown in, Appendix B.

Table 3-4 - Grain Size Distribution Summary – Native Cohesionless Deposit Layer

BH No./ SS No.	Size Fraction (%)					Remarks
	Gravel	Sand	Silt	Clay	Fines	Appendix B
BH-1 / SS3	10	27	-	-	63	Moisture Content (%) = 10
BH-1 / SS9	10	39	38	13	-	Moisture Content (%) = 8
BH-2 / SS4	18	29	38	15	-	Moisture Content (%) = 11
BH-2 / SS7	4	21	-	-	76	Moisture Content (%) = 12
BH-3 / SS3	9	51	-	-	40	Moisture Content (%) = 10
BH-3 / SS10	2	12	-	-	86	Moisture Content (%) = 17

The Standard Penetration Test (SPT) ‘N’ values widely ranged from 23 to 100 blows per 300mm penetration, indicating compact to very dense compactness.

Figure -3 indicate a relation between SPT’N’ values and Elevation, a trend line representing the data is shown in the Figure -3. The trend of the data generally indicates that the cohesionless deposits have a compact compactness (SPT ‘N’ value between 10 and 30) for any proposed footing’s underside elevation between El. 411.50 to El. 412.50 with the exemption of the location of BH-4 where the ground surface elevation is lower, and the soil reaches the compact compactness below El. 409.00.



3.5 GROUNDWATER LEVEL OBSERVATIONS

A monitoring well was installed in borehole BH-1. On November 14, 2023, the groundwater level was measured in the monitoring well at borehole BH-1 and found to be 1.0m below the ground surface (mbgs) at El. **411.4**. The groundwater level was measured upon completion in the monitoring well on October 25, 2023. The recorded groundwater level was 7.3 m below the ground surface at corresponding elevations of 405.1.

It should be noted that the elevations of the recorded groundwater levels are likely to vary throughout the year subject to seasonal variations and extreme weather events.

4.0 GEOTECHNICAL DISCUSSION AND RECOMMENDATIONS

The purpose of the geotechnical investigation was to obtain a geotechnical assessment of the site for the construction of a concrete pad to support the proposed equipment (BESS).

4.1 Ground Characterization and Site Preparation

4.1.1 Frost Depth and Frost Susceptibility

Based on OPSD 3090.101, the Frost Penetration Depth for the project area is 1.6m. Therefore, all foundation elements that are sensitive to movements (i.e., frost-heave and spring-thaw settlements) located in unheated areas should be provided with a minimum of 1.6 metres of non-frost susceptible earth cover or equivalent thermal insulation for frost protection services from the finished grades.

Based on Table 13.1 in the Canadian Foundation Engineering Manual, U.S. Corps of Engineers Frost Design Soil Classification, and since the grain size distribution for the soils intercepted within the frost depth has a percentage of grain sizes, and as such these soils are classified to be type F4 and are classified as high frost susceptibility.

4.1.2 Earthquake Considerations

Based on the bedrock contour maps, the subject site is within the region where the bedrock elevation ranges between elevations 380-400, which indicates an approximate overburden of 16-36 m. Based on the borehole information and according to Table 4.1.8.4. A of the Building Code, the subject site can be classified as Class 'D' for seismic site response. However, it is noted that for code provisions to apply, a 30 m average of soil properties is required, and the available soil data is up to only a maximum depth of 7.9 m in the current investigation. Suppose the Class D seismic site classification should significantly impact the foundation design, then with the undertaking of field shear wave testing, a potential exists for the seismic soil classification to be improved; hence, this is recommended.

4.1.3 Site Grading

At the time of writing this report, no site grading plan was available to us. The finished elevation of the proposed pads is assumed to match the elevation of BH-3 which is **415.0**. Therefore, based on the frost depth of 1.6 m (Section 4.1.1), the Sandy Silt -Sand and Silt Till deposit of high frost susceptibility will be within this depth.

Based on the assumed finish floor elevation of the proposed batteries platform, and based on the current topography of the site, cut / fill operations are highly anticipated, predominantly for any proposed foundation and slab-on-grade support. Engineered fill will be required to establish the design grades and to replace unsuitable subgrade materials.

Portions of the existing earth fill are brown in colour which indicates that the material contains varying amounts of topsoil and organic matter (roots, pieces of wood and glass). Standard Penetration resistance in the earth fill was variable with 'N'-values ranging from 5 to 18 blows per 0.3m. It is generally recognized that earth fill with Standard Penetration Test "N"-values less than 10 blows per 0.3m represent material that has been placed in an inconsistent manner with little or no compaction.

Due to the topsoil and organic inclusions and its non-uniform density, the existing earth fill is unsuitable for supporting structures, slab-on-grade, and pavement in its present condition. In using the existing earth fill for structural backfill or in pavement and slab construction where grade integrity is required, it should be sub-excavated, inspected, sorted free of any appreciable topsoil inclusions and deleterious materials, and properly placed and compacted as engineered fill. If it is impractical to sort the topsoil and other deleterious materials from the earth fill, the fill must be wasted and replaced with properly compacted inorganic clean earth fill provided that the material is environmentally approved and accepted for use. The excavated existing earth fill and native Sand and Silt, Sandy Silt deposits can potentially be used as engineered fill contingent on these soils being separated from material containing excessive amounts of topsoil, organic matter, and debris.

Where the moisture content of the existing earth fill material and excavated native soils are too wet or too dry of their optimum moisture content (OMC) for specified compaction, it will also be necessary to either dry or wet these soils such that they are within approximately 2% of their optimum moisture content for proper compaction.

Once properly conditioned, the excavated site materials should be placed in loose lifts no thicker than 200 mm, and each lift should be compacted to at least 98% of standard Proctor maximum dry density (SPMDD) prior to placing subsequent lifts of material. It should also be noted that the existing earth fill and native soils are frost susceptible and should not be used in locations where frost-related movement would present a concern. The existing earth fill materials and native soils are generally not free draining and will be difficult to handle and compact should they become wetter as a result of inclement weather or seepage.

4.1.4 Engineered Fill

All existing earth fill including wet, disturbed native soils and debris should be excavated and removed from below the proposed batteries platform and pavement areas and replaced with clean fill provided that the soil is environmentally approved and accepted for re-use.

The following recommendations regarding the placement of engineered fill should be adhered to during construction:

Ponding surface water, all topsoil, organic soils, existing earth fill, disturbed loosened/softened materials and any deleterious materials must be removed from below any proposed structures and pavement areas to expose competent subgrade comprised of undisturbed, native soils. Once exposed, the subgrade should be thoroughly compacted and then proof-rolled using heavy rubber-tire equipment under the inspection of qualified geotechnical engineering personnel to detect any soft, weak or unstable soils. Any of this unsuitable subgrade soils found at the time of the proof-rolling that are unable to be uniformly compacted must be removed and replaced by clean aggregate fill material comprised of OPSS Granular ‘B’ Type II material placed in thin, loose lifts (maximum 200 mm thick) and each lift thoroughly compacted to a minimum of 98% of Standard Proctor Maximum Dry Density (SPMDD).

Engineered fill operations should be monitored, and compaction tests should be performed on a full-time basis by a qualified engineering technician supervised by the Geotechnical Engineer.

The boundaries of the engineered fill must be clearly and accurately laid out in the field by qualified surveyors prior to the commencement of engineered fill construction. The top of the engineered fill should extend a minimum of 1.5 m beyond the perimeter of the batteries platform to be supported. Where the depth of engineered fill exceeds 1.5 m, this horizontal distance of 1.5m beyond the perimeter of the structure should be increased by at least 1.0 m for each 1.0 m depth of fill. The edges of the engineered fill should be sloped at a maximum of 3H:1V in order to avoid weakening of the engineered fill edges due to slope movement.

Where engineered fill is placed to raise the grade or replace portions of the subgrade, excavated earth fill, native soils or similar clean imported fill materials approved by qualified geotechnical personnel may be used, provided their moisture content is controlled within 2% of the soil’s optimum moisture content (OMC) as determined by the Standard Proctor test method and the materials are placed in large areas where they can be effectively compacted with heavy Padfoot type rollers. All fills must be placed and compacted in thin, loose lifts (maximum 200 mm thick) to not less than 98% of standard Proctor maximum dry density (SPMDD) before subsequent lifts of material are placed. Excavated existing earth fill material, native sand and silt and sand deposits may be used for grading purposes below the batteries platform and pavement areas where grade integrity is required subject to inspection and approval by qualified geotechnical personnel.

The founding subgrade for footings and underground services must be inspected by the Geotechnical Engineer that supervised the engineered fill construction. This is to ensure that the foundations are placed within the engineered fill envelope and the integrity of the fill has not been compromised by interim construction, environmental degradation and/or disturbance by the footing excavation. Extended footings and/or steel reinforcement may be required based on the footing subgrade inspection.

It needs to be noted that post-construction settlement of compacted fills on the order of 0.5 to 1 percent of total height are common, even when adequately placed to specified compaction. It is best to schedule deep fill placement as far in advance of footing construction and slab-on-grade construction as possible for best grade integrity.

In areas of narrow trenches or confined spaces such as around manholes, catch basins, etc., imported sand or OPSS Granular 'B' should be used and compacted to the specified amount.

4.2 Feasible Foundation Types

4.2.1 Surface Slab on Grade Option

The proposed BESS pads are typically surface slabs (unheated) and exposed to the elements. However, due to the high frost susceptible silty deposit present within the frost depth, the following two options are recommended:

- Option 1: Providing equivalent thermal insulation, this will have to be placed both under a slab and in a perimeter around the slab with a perimeter width equal to the frost depth. This will likely place the slab within the engineered fill explained in Section 4.2.4. However, the durability of thermal insulation is unknown and has the potential to be damaged, for example, due to unintentional trenching in the future.
- Option 2: Sub-excavate to the frost depth of 1.6 m, including an annular perimeter of each slab to a width equal to the frost depth, and replace with OPSS 1010 Type B II. This alternative is more robust approach both from a frost protection and a slab structure bearing support points of view.

For the surface slab on grade option constructed on engineered OPSS 1010 granular backfill in the second option above can be designed for a net geotechnical bearing resistance at Serviceability Limit States (SLS) of 120 kPa, and a factored geotechnical bearing resistance at Ultimate Limit States (ULS) of 180 kPa. The estimated total and differential settlements of footings designed using the recommended SLS bearing resistance should not exceed the conventional limits of 25mm and 19mm. For assessment of base sliding resistance of shallow foundations, an unfactored friction coefficient of 0.48 can be used.

Modulus of Subgrade Reaction (ks) of 15 MPa/m is recommended for slab foundation construction. Minimum reinforcement and provision to accommodate shrink-swell movements of the concrete slab should be addressed by the structural engineer apart from other loading requirements such as use by lifting equipment on the slab.

4.2.2 Shallow Strip or Spread Footings on Engineered Fill Option

Spread footings constructed on suitably prepared engineered fill can be designed for a net geotechnical bearing resistance at Serviceability Limit States (SLS) of 150 kPa, and a factored geotechnical bearing resistance at Ultimate Limit States (ULS) of 225 kPa for vertical loads. The estimated total and differential settlements of footings designed using the recommended SLS bearing resistance should not exceed the conventional limits of 25mm and 19mm, respectively. It should be noted that the bearing resistance values could be less than stated due to eccentric loading conditions. The foundation design must consider load inclinations and eccentricity as per the applicable principles presented in the Canadian Foundation Engineering Manual (CFEM).

All exterior footings and footings in unheated areas should be provided at least 1.6m of soil cover or equivalent artificial thermal insulation for frost protection purposes. Exposed soil foundation subgrades at the time of construction should be protected against freezing at all times until sufficient backfill is placed within the foundation areas for frost protection. Surface water should be kept away from the foundation subgrade areas to prevent softening. If unstable subgrade conditions develop, AME should be contacted in order to assess the conditions and make appropriate recommendations.

Due to variations in the founding soil strength and / or loosening caused by excavating disturbance and / or seasonal frost effects, all footing subgrades must be evaluated by the Geotechnical Engineer prior to placing formwork and foundation concrete to ensure that the soil exposed at the excavation base is consistent with the design geotechnical bearing resistance.

The quality of the subgrade must be inspected by the Geotechnical Engineer during construction, prior to constructing the footings and placing the footing concrete, to confirm that the founding subgrade is consistent with the design bearing requirements.

Prior to pouring concrete for footings, the footing bases must be cleaned of all deleterious materials such as topsoil, earth fill, softened, disturbed, or caved materials, as well as any standing water. If construction proceeds during freezing weather conditions, adequate temporary frost protection for the footing base and foundation concrete must be provided.

4.2.3 Shallow Strip or Spread Footings on Native Soil Option

At the locations within the vicinity of boreholes BH-1, BH-2 and BH-3, where the underside of any proposed shallow foundation between El. 411.50 to El. 412.50 and considering the compact compactness of the native cohesionless till as encountered in the vicinity of the proposed BESS a factored geotechnical resistance at ULS of 375 kPa and a geotechnical reaction at SLS of 250 kPa (for 25 mm of settlement) may be used for the foundation design.

The foundation design must consider load inclinations and eccentricity as per the applicable principles presented in the Canadian Foundation Engineering Manual (CFEM).

The geotechnical resistances and settlement depend on embedment depth, footing size and applied loads. The geotechnical resistance provided above should be reviewed if the selected footing width or founding elevation differs from those provided above.

Footings in exterior and unheated interior areas are required to be 1.6m below the ground surface for frost protection.

4.2.4 Helical Piles Option

The design of helical piles should conform to Canadian Foundation Engineering Manual (CFEM). The axial compression capacity of helical piles should be confirmed in the field by pile testing.

As an indicative loading capacity estimate, professionally designed, constructed and proof load tested helical piles (square, slender shaft, triple helix with 300 mm diameter upper helix) embedded at least 1.5 m below El. 411.00 in the dense native cohesionless deposit should have a working capacity at least 250 kN in axial compression.

Helical piles are generally designed and constructed by specialist contractors. Shop drawings should be submitted to McIntosh Perry to ensure compliance with this geotechnical report. The installation of helical piles should be inspected by a qualified Geotechnical Engineer to document pile size, pile toe depth, and installation torque with depth.

4.2.5 Perimeter Drainage

Due to the high susceptible soils and relatively shallow groundwater elevation at BH-1 (411.4 masl) encountered on this site, a perimeter drainage is recommended for slab-on-grade with continuous/spread footings.

4.3 Excavations and Backfill

4.3.1 Excavatability Issues

Excavation of overburden soil can be performed using conventional hydraulic excavating equipment. The existing monitoring well needs to be decommissioned as per applicable regulations to depths below any planned excavation levels.

In view of the above, provision must be made in the excavation contract for the removal of possible boulders in the till, obstructions in the fill and abandoned surface/buried structures.

4.3.2 Open-Cut Excavation Stability

Temporary excavations for the shallow foundations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA).

Excavation for the proposed shallow foundations wall may extend to a depth 1.6 m below the existing ground surface.

According to OHSA, if the excavation is deeper than 1.2 m, the excavation sides should be sloped. The slope of the sides depends on the type of excavated materials. Table 4-1 stipulates the maximum slopes of excavation for various soil types.

Table 4-1: Soil Type and Slope Ratio

Soil Type	Base of Slope	Maximum Slope Inclination
1	Within 1.2 meters of bottom of trench	1H:1V
2	Within 1.2 meters of bottom of trench	1H:1V
3	From bottom of trench	1H:1V
4	From bottom of trench	3H:1V

- Fill soil may be classified as Type 4 soil.
- The angle of repose of the sandy soil should be considered in excavations in dry and or wet sandy soil. The estimated angle of repose of the sandy soil is approximately 28°.
- Excavation of side slopes should be protected from exposure to precipitation and associated ground surface runoff and should be inspected regularly for signs of instability. If localized instability is noted during excavation or if wet conditions are encountered, the side slopes should be flattened as required to maintain safe working conditions. Furthermore, we recommend covering the side slopes with plastic sheets to minimize/prevent the sandy soil from sloughing and/or caving during rainy seasons.
- Construction traffic and stockpiles of soils be kept away from the edges of the excavations for equal to the depth of the excavation. if such clearance cannot be maintained, the resulting surcharge loads should be considered in the design of the shoring system. In all cases, OHSA and other regulatory requirements must be followed, and adequate protection provided for workers.
- It should be noted that the above recommendations are based on the actual borehole locations.

- The existing native sandy /silty sand may be used for construction backfilling, provided that these materials are free from organics and any other deleterious materials.
- The contractor should be aware of all existing utility locations before excavating and should be prepared to support/brace or relocate them as required.

4.3.3 Temporary Shoring

Walls supporting the soil should be designed and constructed to resist lateral earth pressure imposed by the soil and hydrostatic pressure. For walls that are designed to allow rotation, active earth pressures may be used for design. For rigidly tied walls, the "at rest" earth pressure should be used for design. The lateral earth pressure may be calculated using the following expression:

$P = K\,[\gamma(\eta - \eta_\omega) + \gamma' \eta_\omega + \vartheta] + \gamma_\omega \eta_\omega$

The above equation considers the hydrostatic pressure in determining the lateral pressure on the wall if continuous wall drains are not provided.

Perimeter drainage system may be constructed to prevent the build-up of any hydrostatic pressure behind the wall. In this case, the following equation is recommended:

$P = K\,(\gamma\eta + \vartheta)$

Summary of soil parameters is provided in Table 4-2.

Table 5-2 Soil Parameters

Soil Parameters	$Ø^\circ$	C (kPa)	γ (kN/m^2)	K_A	K_0	K_P
Compact Sandy Silt	30	0	19	0.33	0.5	3.0

Table 5-3 Description of Soil Parameters

Soil Parameter	Description
P	Lateral earth pressure in kPa acting at depth h (m)
Ø	Internal angle of friction
γ	Unit weight of backfill, a value of 20 kN/m^3 may be assumed
γ_ω	Unit weight of water, a value of 10 kN/m^3 may be assumed
γ'	Submersed unit weight of backfill, a value of 10 kN/m^3 may be assumed
h	Depth to point of interest in meters
h_w	The depth below the groundwater table (m)
q	Equivalent value to surcharge on the ground surface in kPa.
K	Lateral earth pressure coefficient (horizontal ground surface and vertical wall
K_A	Active earth pressure coefficient
K_0	At rest earth pressure coefficient
K_P	Passive earth pressure coefficient

4.4 Buried Utilities and Service Trenches

4.4.1 Sub-Grade Considerations

Based on the existing site grades, sewer pipes and electrical conduits will most likely be supported on the undisturbed native soils. The above-described native soils at the Site are suitable for supporting utilities, sewer pipes, etc.

Maintenance hole chambers, valve chambers and other related access chambers associated with the proposed service installations that are of structural concrete can be supported on the native silty clay till deposit, the structural utilities can be founded on pad foundations with a factored ULS of 150 kPa and a SLS of 100 kPa for total settlements not exceeding 25 mm.

Careful preparation and strengthening of the trench bases before sewer installation will be required if unsuitable bedding conditions occur. The subgrade may be strengthened by placing a thick mat consisting of 50-mm crusher-run limestone. Field conditions will determine the depth of the stone required. Geotextiles and/or geogrids may also be used.

4.4.2 Pipe Bedding, Cover and Backfill

Construction should be carried out in accordance with OPSS.MUNI 401 “Trenching, Backfilling, and compacting”. All bedding, cover and backfill materials should be selected according to OPSS 1010 Aggregates – Base, Subbase, Select Sub-grade, and Backfill Material.

4.4.3 Pipe Bedding and Cover

Pipe sections should be placed with a minimum bedding thickness as prescribed by OPSD 802.032 for rigid pipes and OPSD 802.010 for flexible pipes. These minimum thicknesses are to be confirmed based on prevailing conditions at the time of construction. Normal Class 'B' type bedding is recommended for underground utilities. OPSS 1010 Granular ‘A’ or 19 mm crusher-run limestone can be used as bedding material.

The bedding material is to be placed in layers not exceeding 300 mm in thickness and should be compacted to a minimum of 98% SPMDD. Bedding on each side of the pipe should be completed simultaneously, and at no time are the levels on each side to differ by more than the 300 mm.

4.4.4 Trench Backfill

Trench backfilling should be carried out immediately following trench excavation and service installation. Sand cover material should be placed as backfill to at least 600 mm above the crown of pipes.

Backfill shall be placed to a minimum height of 600 mm or subject to the pipe manufacturer’s specifications above the crown of the pipe before power-operated equipment is used for compacting. Compaction should be carried out to at least 95% of the material’s SPMDD.

It should be noted that achieving compaction of the backfill may become difficult where backfilling in confined areas such as beneath an existing pipe or service crossing within the trench area. In such circumstances the accepted practice is to use Un-shrinkable Fill (U-Fill) to backfill the trench. This will ensure that the trench has been backfilled such that support for the existing pipe or service crossing has been reinstated to match or exceed the previous sub-grade condition.

5.0 GENERAL CONSTRUCTION CONSIDERATIONS

All grades raise general fill (if required) should conform to OPSS 212 – Construction Specification for Earth Borrow.

Temporary frost protection should be provided to protect subgrade materials from freezing if construction is carried out under winter conditions. Similarly, for any embankment fill construction during winter periods, inspection should be undertaken daily to identify any frost affected top thickness and if found should sub-excavated and discarded, to be replaced with engineered sound fill material.

Temporary erosion and sediment control measures during construction should conform to OPSS.MUNI 805.

The excavated material from excavations should be checked for contamination to determine which disposal option is best suited for the excavated materials (OPSS.MUNI 180), prior to removal/ disposal off-site, if required to be disposed of. The investigation of disposal requirements is outside MPCE’s geotechnical scope.

6.0 IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

When writing this report, no detailed design and final elevations were available. Hence the recommendations given in this report are very broad and limited to the construction of the BESS pad. Therefore, further recommendations related to the construction of other elements must be confirmed based on additional foundation investigations on a site-specific basis for proposed constructions in the future.

The geotechnical assessment presented in this report are intended for the sole guidance of the client named and their design consultants. It should not be relied upon for any other purpose.

The information on which these recommendations are based is subject to confirmation by MPCE geotechnical engineering personnel at the time of construction.

The data we have collated and the opinions we have formed after reviewing this information should not be construed as a guarantee but only as a guide to probable expectations. Conditions that exist, but are not recorded herein, were not apparent given the level of study authorized.

Localized variations in the subsurface conditions may be present between and beyond the boreholes advanced, and that these conditions may be significantly different from the general description provided for design purposes.

It is strongly urged that MPCE should be contacted to aid in the interpretation of the borehole records by anyone undertaking work on/or below the ground surface at this Site prior to this work being carried out.

The client expressly agrees that it has entered into this agreement with MPCE, both on its own behalf and as an agent on behalf of its employees and principals.

The client expressly agrees that MPCE employees and principals shall have no personal liability to the client in respect of a claim, whether in contract, tort, and/or any other cause of action in law. Accordingly, the client expressly agrees that it will bring no proceedings and take no action in any court of law against any MPCE employees or principals in their personal capacity.

7.0 CLOSURE

We trust that the following information is sufficient for your needs. We will be pleased to discuss the salient findings of this report with you, should you wish. If you require our further services in this regard, please do not hesitate to contact our office.

Yours truly,

McIntosh Perry Consulting Engineers Ltd.

Field work carried out by:

____

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____

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McINTOSH PERRY

APPENDIX A

Borehole Location Plan and
Borehole Logs

Borehole Locations

6234 Guelph Rd, Centre Wellington, ON.

McINTOSH PERRY

PROJECT NO.: **CCI-242306-00**

PROJECT: Geotechnical Investigation

CLIENT: McIntosh Perry Consulting Engineers

PROJECT LOCATION: 6235 Guelph Road, Centre Wellington, Ontario

Drilling Date: Oct/25/2023 - Oct/25/2023

BH Location: N 551940.193 E 4837035.372

Drilling Equipment: M5T

Drilling Method: Solid Stem Auger

Remarks:

BH No: BH-1

Datum: Geodetic

Elevation: 412.401 m asl

Compiled by: PS

Checked by: ZB

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	Water Content (%)	Remarks and Grain Size Distribution (%) GR SA SI CL
412.4 / 0.0	**TOPSOIL**	1	SS	3	100%		
412.1 / 0.4	**FILL** sandy silt, some clay, trace of gravel, roots & topsoil, dark brown, moist						
411.5 / 0.9	**SANDY SILT TILL** some clay, trace of gravel, brown, moist, compact to dense to very dense	2	SS	18	100%		
		3	SS	29	40%	10	10 27 (63)
		4	SS	55	90%		
		5	SS	80	100%		
	dense to very dense below 2.4m.	6	SS	81	100%		
408.1 / 4.3	**SAND AND SILT TILL** trace of clay & gravel, brown, moist, very dense	7	SS	100	40%		
		8	SS	100	100%		
		9	SS	100	100%	8	10 39 38 13
405.2 / 7.2	**SANDY SILT TILL** trace of clay & gravel, grey, moist, very dense	10	SS	60	100%		
		11	SS	100	100%		
404.5 / 7.9	**END OF BOREHOLE** 1) Borehole was open and WL at 7.32m on completion.						

Groundwater: W. L. 411.4 Nov 14, 23

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80

SHEAR STRENGTH (kPa): Field. Shear Vane (x) & Sensitivity (s); Pocket Penetrometer ✖; ● Quick Triaxial; ○ Unconfined; 20 40 60 80

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30 40 50 60 70 80 90

Unit Weight (kN/m^3) Pocket Penetro. (kPa)

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity. ○ 8=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCI-242306-00**

PROJECT: Geotechnical Investigation

CLIENT: McIntosh Perry Consulting Engineers

PROJECT LOCATION: 6235 Guelph Road, Centre Wellington, Ontario

Drilling Date: Oct/25/2023 - Oct/25/2023

BH Location: N 551828.657 E 4837103.147

Drilling Equipment: M5T

Drilling Method: Solid Stem Auger

Remarks:

BH No: BH-2

Datum: Geodetic

Elevation: 416.072 m asl

Compiled by: PS

Checked by: ZB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	DEPTH	ELEVATION	WATER CONTENT (%)	GR	SA	SI	CL
416.1						0.0	416					
0.0	**TOPSOIL**											
415.8 / 0.2	**FILL** sandy silt, some clay, trace of gravel, roots & topsoil, dark brown, moist.	1	SS	6	100%							
		2	SS	22	35%	1.0	415	11	10	35	43	12
		3	SS	18	40%	2.0	414					
413.9 / 2.1	**SANDY SILT TILL** trace of clay & gravel, brown, moist, compact	4	SS	47	90%			11	18	29	38	15
	- - - - - - - - - - - - - some clay, very dense	5	SS	50	100%	3.0	413					
		6	SS	50	100%	4.0	412					
	- - - - - - - - - - - - - trace of clay, dense at 4.5m.	7	SS	38	100%	5.0	411	12	4	21	(74.5)	
	- - - - - - - - - - - - - trace of clay, greyish brown, moist, very dense.	8	SS	100	100%							
		9	SS	100	100%	6.0	410					
		10	SS	100	20%	7.0	409					
408.4		11	SS	100	80%							
7.7	**END OF BOREHOLE** 1) Borehole was open and dry on completion.											

GRAPH NOTES

30 Upper value = Field Vane Shear Strength ○ ❽=3%

3 Lower value = Vane Sensitivity Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCI-242306-00**

PROJECT: Geotechnical Investigation

CLIENT: McIntosh Perry Consulting Engineers

PROJECT LOCATION: 6235 Guelph Road, Centre Wellington, Ontario

Drilling Date: Oct/26/2023 - Oct/26/2023

BH Location: N 551695.406 E 4837195.639

Drilling Equipment: M5T

Drilling Method: Solid Stem Auger

Remarks:

BH No: BH-3

Datum: Geodetic

Elevation: 414.987 m asl

Compiled by: PS

Checked by: ZB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)	Natural Moisture Content (%)	Remarks and Grain Size Distribution (%) GR SA SI CL
415.0 / 0.0	**TOPSOIL**						
414.7 / 0.3	**FILL** silty sand, trace of clay, gravel, roots & topsoil, dark brown, moist.	1	SS	3	70%		
414.1 / 0.9	**SAND AND SILT TILL** trace of clay and gravel, brown, moist, compact to dense	2	SS	16	80%		
		3	SS	23	100%	10	9 51 (40)
		4	SS	47	90%		
412.0 / 3.0	**SANDY SILT TILL** trace of clay and gravel, brown, moist, very dense	5	SS	57	80%		
		6	SS	100	90%		
		7	SS	100	100%		
		8	SS	100	100%		
408.9 / 6.1	**SILT TILL** trace to some sand and clay, trace of gravel, grey, moist, very dense	9	SS	69	90%		
		10	SS	50	90%	17	2 12 (86)
		11	SS	100	100%		
407.2 / 7.8	**END OF BOREHOLE** 1) Borehole was open and dry on completion.						

Chart columns: GROUNDWATER CONDITIONS; DEPTH / ELEVATION (0.0; 1.0 414; 2.0 413; 3.0 412; 4.0 411; 5.0 410; 6.0 409; 7.0 408); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80); SHEAR STRENGTH (kPa) — Field. Shear Vane (x) & Sensitivity (s), Pocket Penetrometer ✖, ● Quick Triaxial, O Unconfined (20 40 60 80); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30 40 50 60 70 80 90; Unit Weight (kN/m³) Pocket Penetro. (kPa).

GRAPH NOTES: 30/3 Upper value = Field Vane Shear Strength; Lower value = Vane Sensitivity. O 8=3% Strain at Failure

McINTOSH PERRY

PROJECT NO.: **CCI-242306-00**

PROJECT: Geotechnical Investigation

CLIENT: McIntosh Perry Consulting Engineers

PROJECT LOCATION: 6235 Guelph Road, Centre Wellington, Ontario

Drilling Date: Oct/26/2023 - Oct/26/2023

BH Location: N 551617.676 E 4837328.77

Drilling Equipment: M5T

Drilling Method: Solid Stem Auger

Remarks:

BH No: BH-4

Datum: Geodetic

Elevation: 411.858 m asl

Compiled by: PS

Checked by: ZB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" BLOWS/0.3 m / RQD (%)	RECOVERY (%)
411.9 / 0.0	**TOPSOIL**	1	SS	5	100%
411.4 / 0.5	**FILL** sandy silt, some clay, trace of gravel & topsoil, dark brown, very moist.	2	SS	10	60%
		3	SS	10	100%
		4	SS	12	80%
409.2 / 2.7	**SANDY SILT TILL** trace of clay and gravel, brown, moist, compact to very dense	5	SS	35	80%
		6	SS	100	50%
		7	SS	100	30%
		8	SS	67	100%
		9	SS	100	90%
		10	SS	100	60%
		11	SS	100	70%
404.1 / 7.7	**END OF BOREHOLE** 1) Borehole was open and dry on completion.				

Column headings: SOIL PROFILE; SAMPLES; STRATA PLOT; GROUNDWATER CONDITIONS; DEPTH; ELEVATION; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80; SHEAR STRENGTH (kPa) Field. Shear Vane (x) & Sensitivity (s) Pocket Penetrometer ✖ ● Quick Triaxial ○ Unconfined 20 40 60 80; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30 40 50 60 70 80 90; Remarks and Grain Size Distribution (%) Unit Weight (kN/m^3) Pocket Penetro. (kPa) GR SA SI CL

Depth: 0.0, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0

Elevation: 411, 410, 409, 408, 407, 406, 405

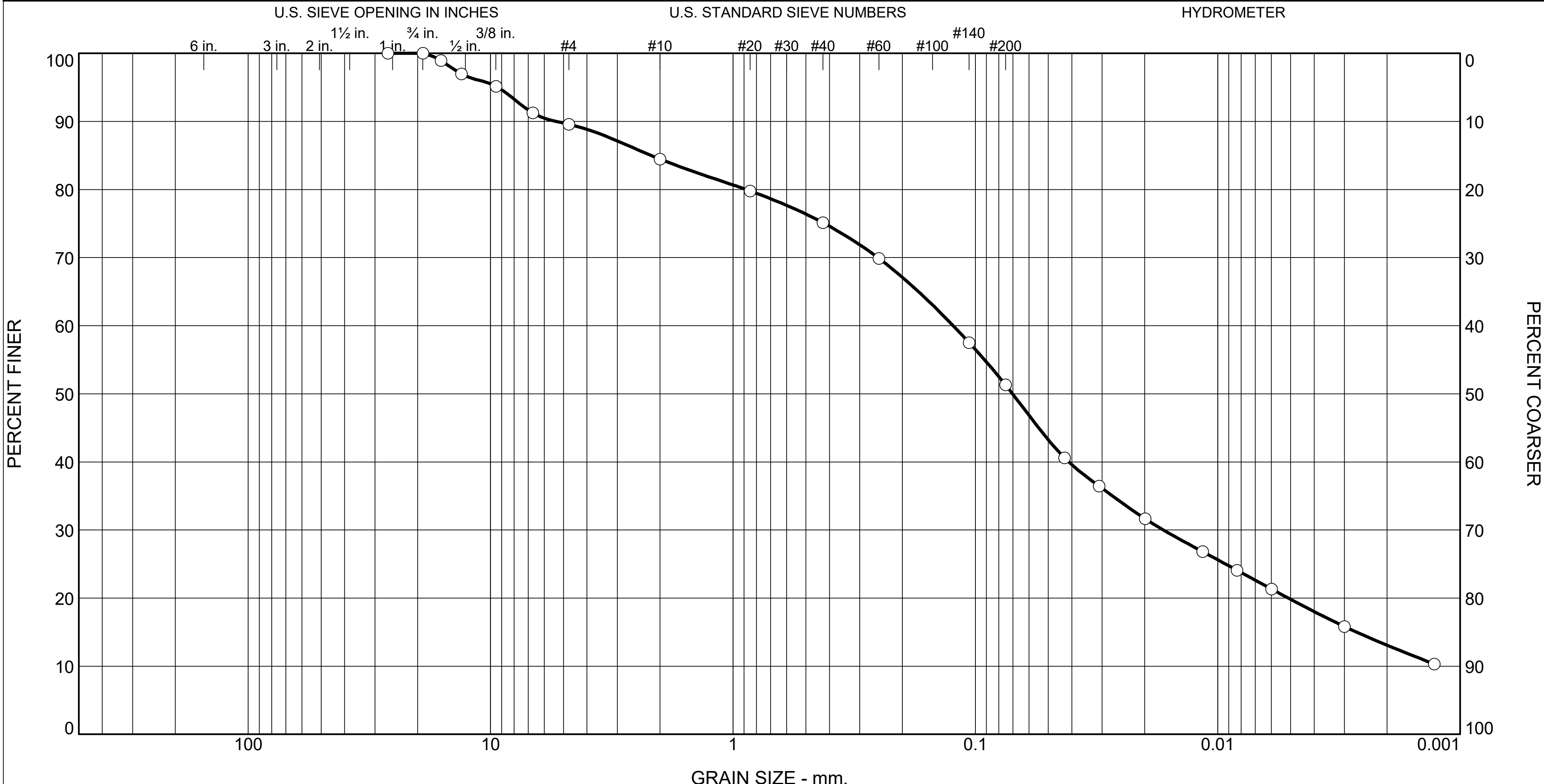
GRAPH NOTES: $\overset{30}{\underset{3}{\times}}$ Upper value = Field Vane Shear Strength, Lower value = Vane Sensitivity; ○ 8 =3% Strain at Failure

1MP SOIL LOG CCO-242294-00_6235 GUELPH ROAD, CENTRE WELLINGTO, ON_BOREHOLE LOG - GINT(DRAFT) - NOV 15, 2023.GPJ MP_OTTAWA_FOUNDATIONS.GDT 11/15/23

McINTOSH PERRY

APPENDIX B
Laboratory Tests

Particle Size Distribution Report



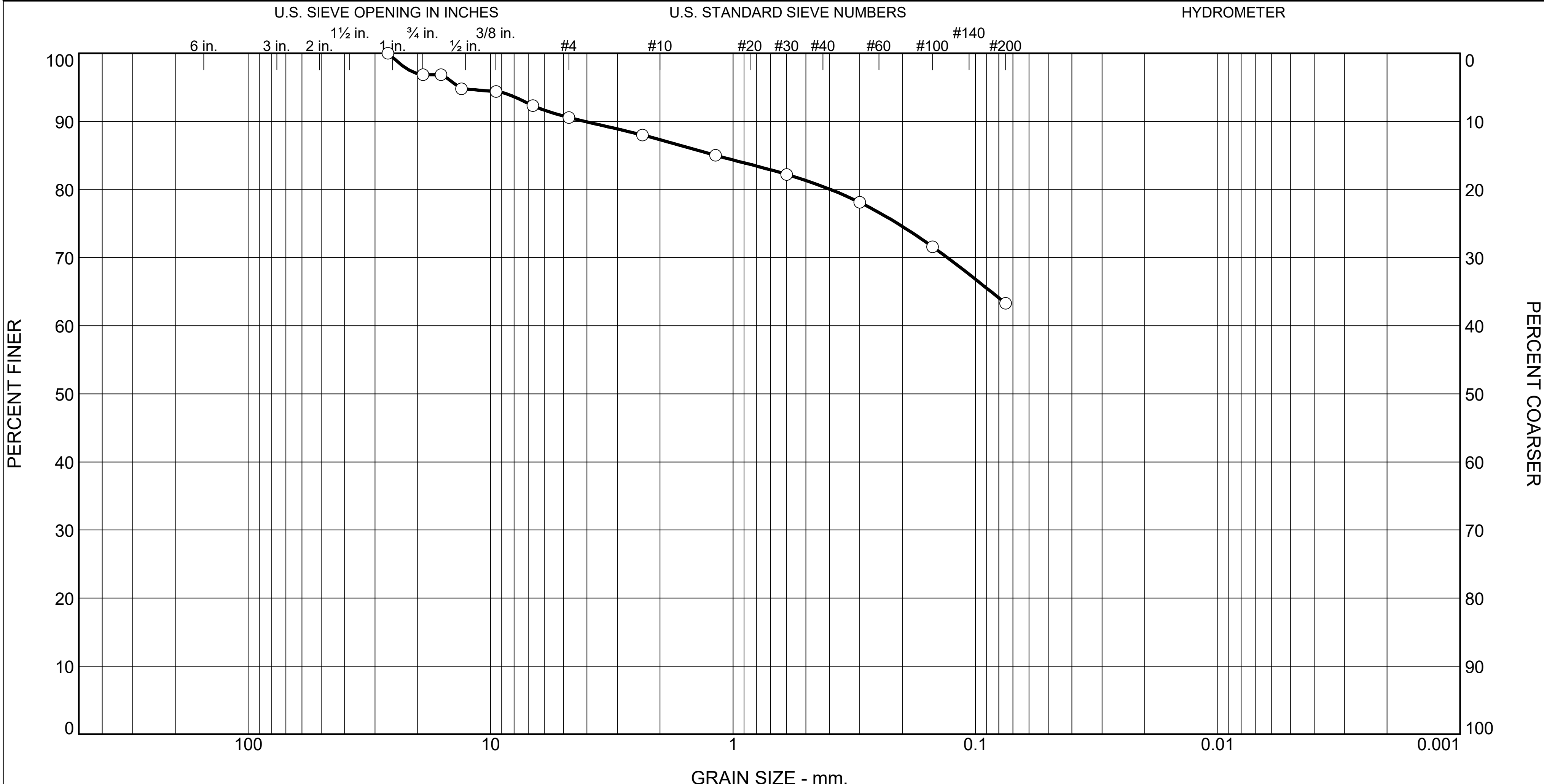
	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	10.4	5.1	9.4	23.8	38.2	13.1

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH# 1 SS9			Nov 5, 2023

Client		**McIntosh Perry**	○ F.M.=1.36
Project 6235 Guelph Road, Centre Wellington, Ontario			
		Vaughan, Ontario	
Project No. CCO-242306-00	Figure		

Tested By: D.S

Particle Size Distribution Report



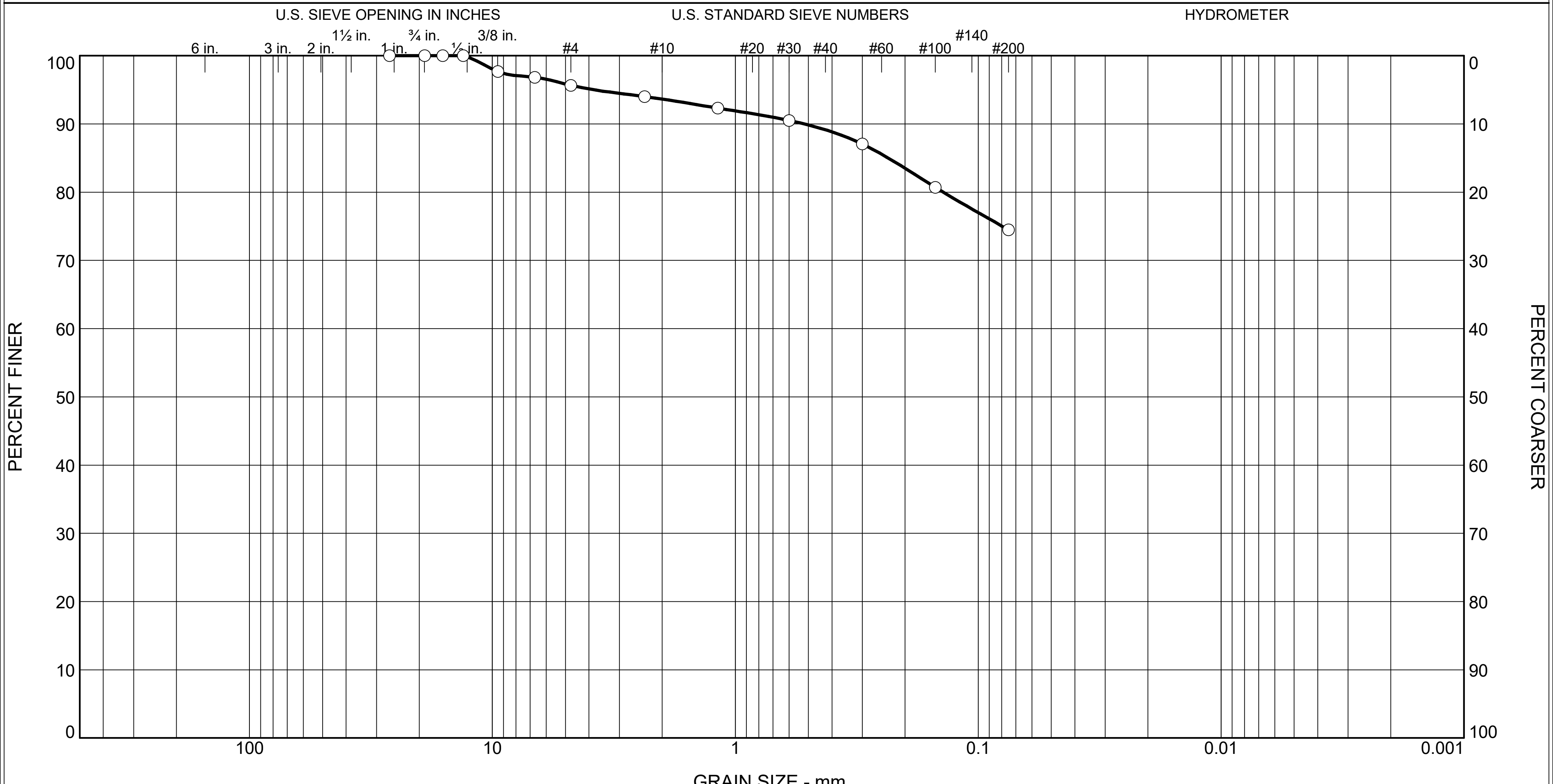
	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	3.1	6.3	3.3	6.9	17.1	63.3	

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH#1 SS 3			November 5,

Client	**McIntosh Perry**	○ F.M.=1.13
Project 6235 Guelph Road, Centre Wellington, Ontario		
	Vaughan, Ontario	
Project No. CCO-242306-00 / Figure		

Tested By: D.S

Particle Size Distribution Report



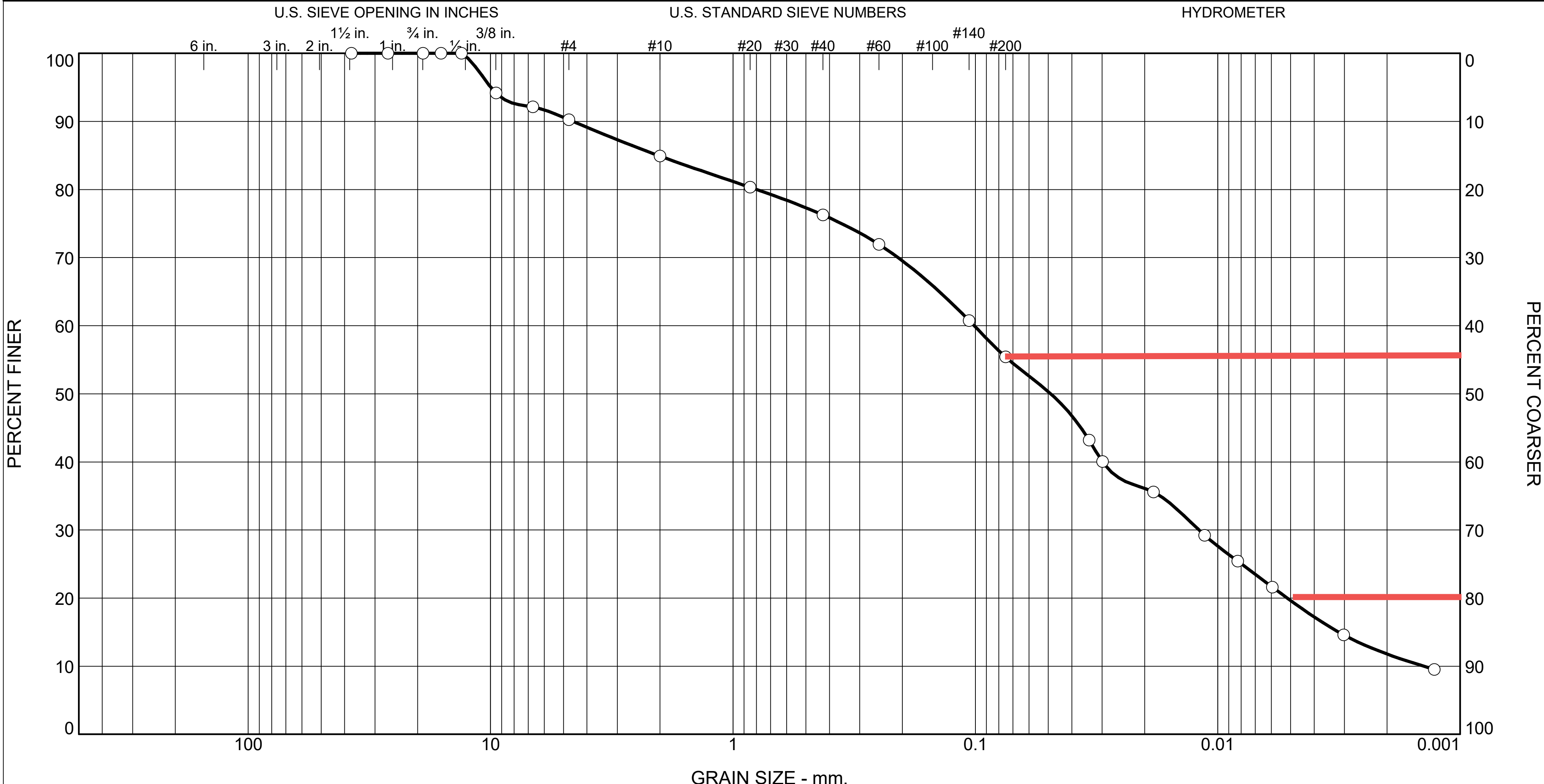
	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	4.3	2.1	4.5	14.6	74.5	

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH# 2 SS 7			November 5,

Client	**McIntosh Perry** **Vaughan, Ontario**	○ F.M.=0.62
Project 6235 Guelph Road, Centre Wellington, Ontario		
Project No. CCO-242306-00 Figure		

Tested By: D.S

Particle Size Distribution Report



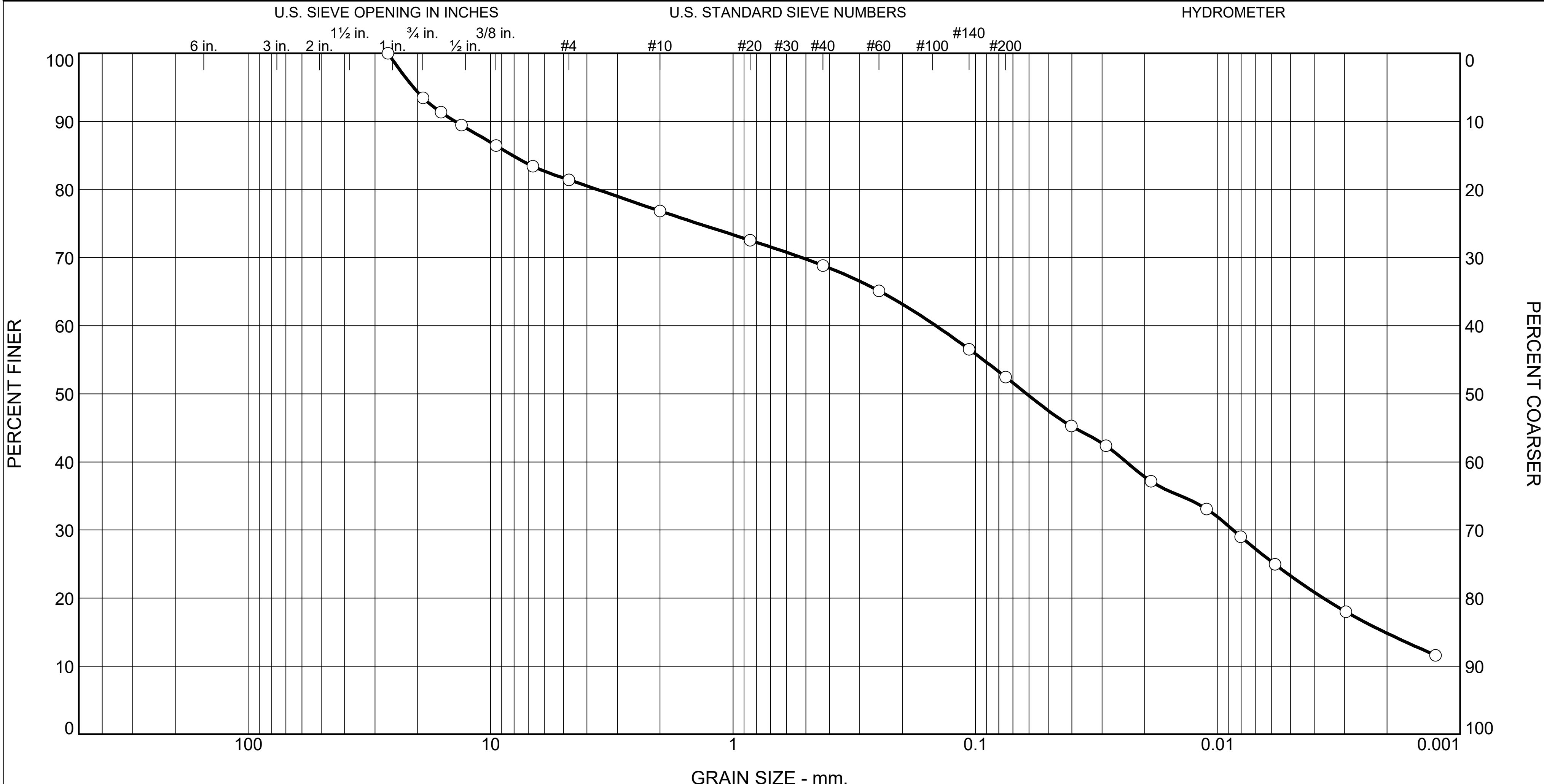
	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	9.7	5.4	8.6	20.9	43.6	11.8

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH#2 SS 2			November5, 2023

Client	**McIntosh Perry**	○ F.M.=1.30
Project 6235 Guelph Road, Centre Wellington, Ontario		
	Vaughan, Ontario	
Project No. CCO-242306-00 Figure		

Tested By: D.S

Particle Size Distribution Report



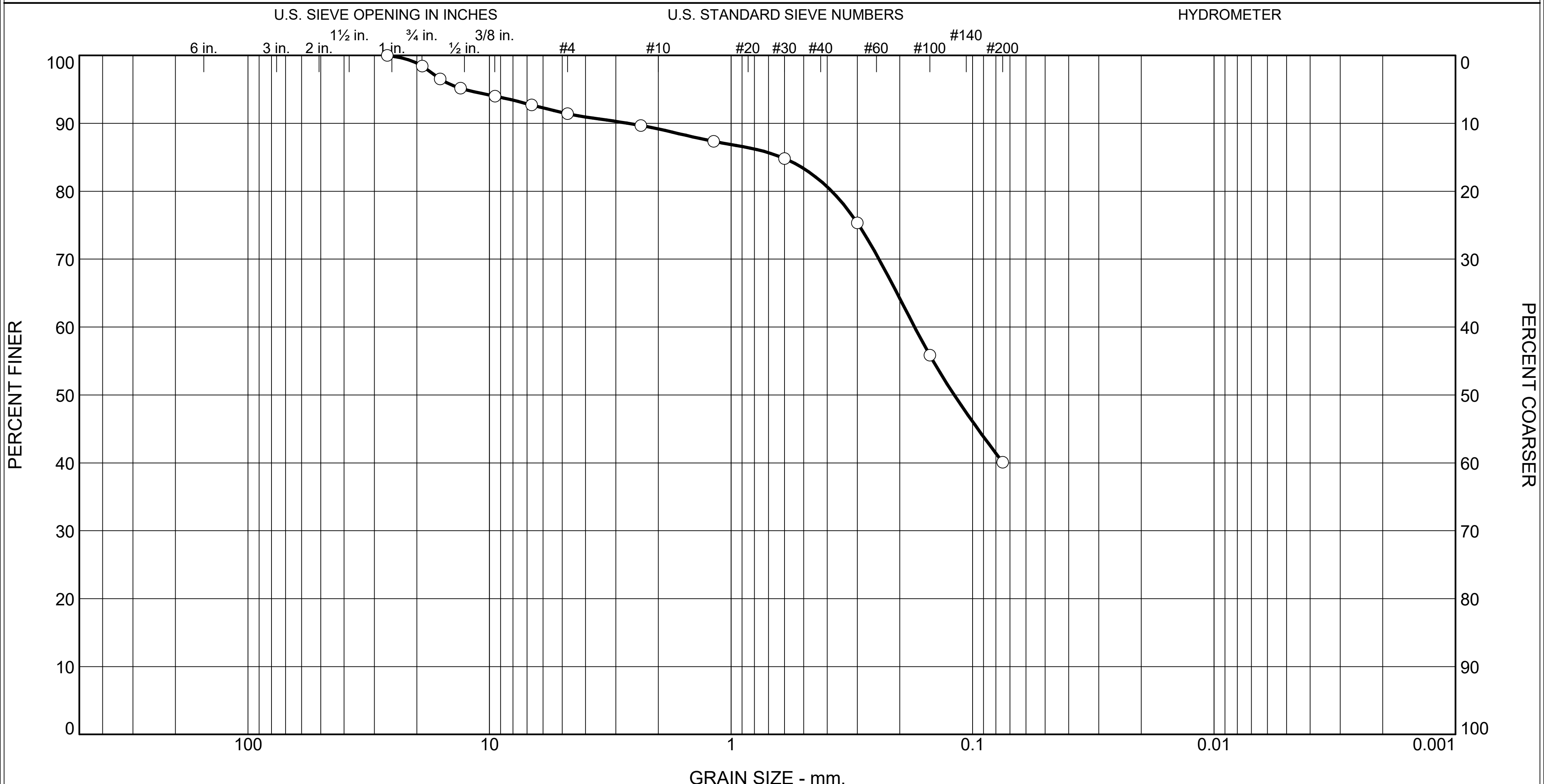
	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	6.5	12.1	4.6	8.0	16.3	37.7	14.8

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH#2 SS 4			November6, 2023

Client	**McIntosh Perry**	○ F.M.=1.89
Project 6235 Guelph Road, Centre Wellington, Ontario		
	Vaughan, Ontario	
Project No. CCO-242306-00 Figure		

Tested By: D.S

Particle Size Distribution Report



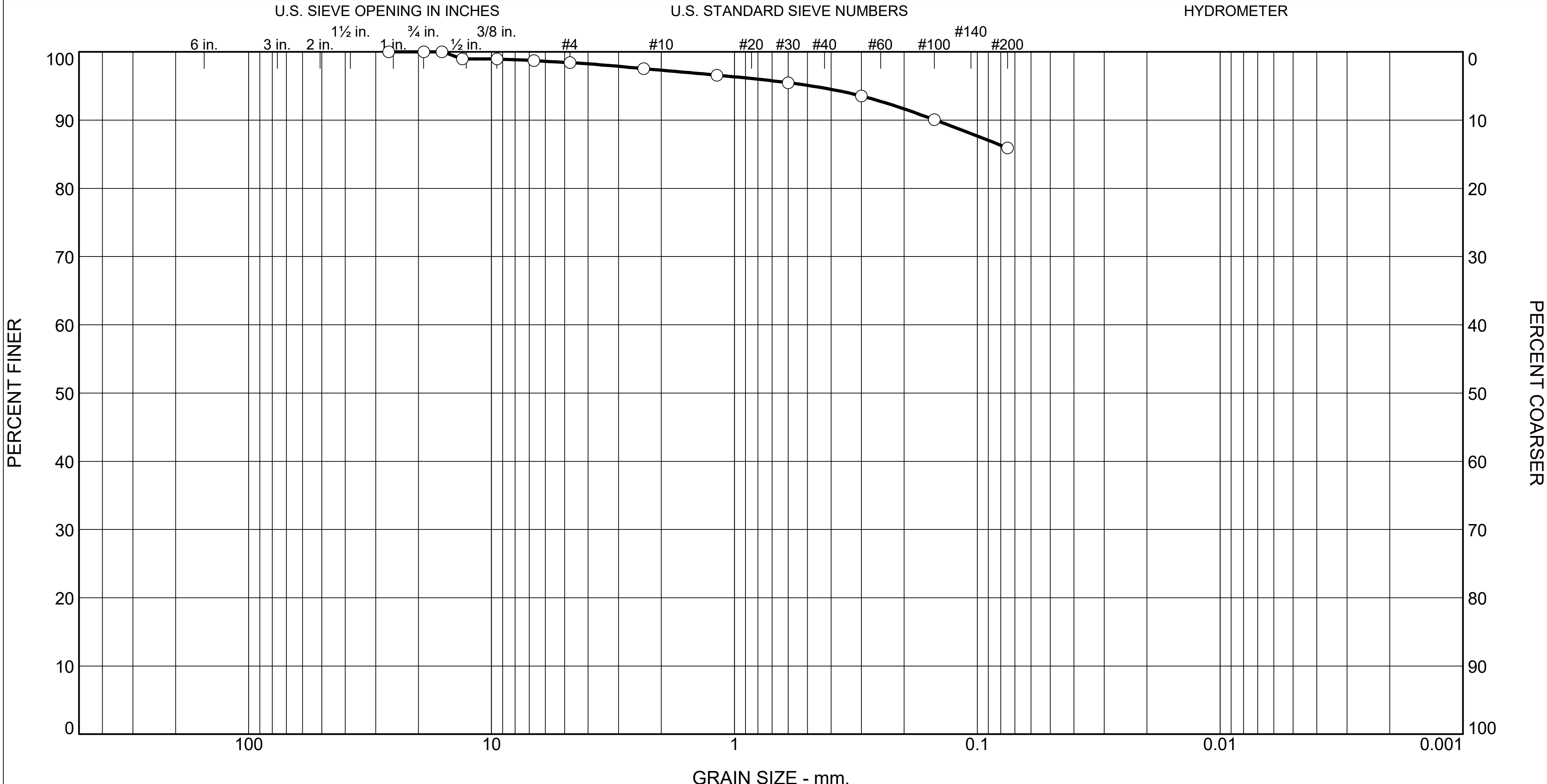
	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	1.5	7.1	2.2	7.7	41.4	40.1	

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH#3 SS 3			Nov5, 2023

Client	**McIntosh Perry**	○ F.M.=1.23
Project 6235 Guelph Road, Centre Wellington, Ontario		
	Vaughan, Ontario	
Project No. CCO-242306-00 / Figure		

Tested By: D.S

Particle Size Distribution Report



	% +75mm	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	1.6	1.1	2.6	8.8	85.9	

Identification	Date Sampled	Date Received	Date Tested
Sample Number: BH# 3 SS 10			November 5,

Client	**McIntosh Perry**	○ F.M.=0.29
Project 6235 Guelph Road, Centre Wellington, Ontario		
	Vaughan, Ontario	
Project No. CCO-242306-00 Figure		

Tested By: D.S