Geotechnical Investigation Report

65 Northey's Bay Road, North Kawartha, ON

December 3, 2025

Prepared for:
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In Association With:
EcoVue Consulting Services Inc.

Cambium Reference: 15101-001 Rev.3

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1.0 Introduction

Cambium Inc. (Cambium) was retained by Eric Challenger (The Client) to conduct a geotechnical investigation and provide geotechnical engineering design advice for the proposed residential development of the property located at 65 Northey's Bay Road in the Township of North Kawartha, Ontario. A Site Location Plan is provided as Figure 1.

It is currently planned for the existing property to be subdivided into 58 new residential lots, one commercial lot, and one block for stormwater management. Associated residential roadways and driveways will be constructed as part of the development. The remaining sections of the property will remain as open space.

It is assumed that single family low rise residential dwellings will be constructed on each of the lots. The homes will have, at most, one basement level, and that basements will extend up to a maximum of 1.8 meters below surface grade (mbgs). The lots will each be privately serviced with a groundwater well and septic system.

Cambium is completing a hydrogeological assessment for the water supply and wastewater for the site. The results of the assessment are provided in another report under a separate cover.

2.0 Site Description

The site is an irregular shaped parcel of land located on the east side of Northey's Bay Road in the Township of North Kawartha, Ontario. The site is located south of Highway 28, bordering Otis Northey Road to the north, Northey's Bay Road to the west, and undeveloped land to the south and east.

Preliminary development plans were provided to Cambium as part of this investigation:

- EcoVue Consulting Services Inc. (2025, March 12). *DP1 Draft Plan: Woodview Golf, 65 Northey's Bay Road, Part of Lots 6 and 7, Concession 6, Southern Division, Geographic Township of Burleigh, Township of North Kawartha, County of Peterborough*. Project No. 21-2207.

The property is currently developed with the Woodview Golf Course. It is understood that the golf course has been recently closed. The residential subdivision will be constructed over the golf course and the proposed commercial severance will be located in the area of the entrance parking lot. The existing structures will be removed accordingly as part of the development.

3.0 Methodology

3.1 Subsurface Investigation – Test Pits

A test pit investigation was conducted at the site on November 4, 2022, to assess subsurface conditions. A total of 33 test pits (numbered TP101-22 through TP133-22) were advanced across the site. Locations of the test pits relative to existing and proposed conditions are shown on Figure 2 – Test Pit Location Plan. Test pits all terminated due to practical refusal on the underlying bedrock and were advanced to depths ranging from surface refusal to 1.4 mbgs.

Soil samples were inspected and logged in the field using visual and tactile methods. The samples were placed in labelled plastic containers for transport and sent to our geotechnical laboratory for review by a senior geotechnical engineer, physical laboratory testing, and temporary storage. The test pits were backfilled with the excavated materials following completion.

The prepared test pit logs are provided in Appendix A. Site soil and groundwater conditions and our geotechnical recommendations are presented in the following sections of this report.

The test pits were excavated using a rubber-tired backhoe under the supervision of a Cambium technician. Dynamic Probe Penetration Testing (DPT) was completed in 15 test pit locations (TP101-22 through TP103-22, TP108-22, TP116-22 through TP119-22, TP124-22 through TP129-22, and TP132-22) from the surface to practical refusal values. The results were recorded for the sampled intervals as the number of blows required to drive a 19 mm diameter steel rod into the soil with an 8 kg hammer falling 750 mm. The DPT values are used in this report to assess consistency of cohesive soils and relative density of non-cohesive materials. Consistency and relative density are based on approximate Standard Penetration testing results: SPT N-values. The correlation between DPT values and consistency/relative density are based on empirical correlations of N-values from soils of similar composition. The described consistency/relative density in this report should be considered approximate.

GPS coordinates of each test pit were obtained using a handheld GPS device. The elevation of each borehole was determined relative to a site benchmark, a steel nail at the base of road sign located at the entrance of the site, which was assigned a relative elevation of 100 m.

3.2 Laboratory Testing

Laboratory soil testing included seven Particle Size Distribution Analyses (LS 702) and Natural Moisture Content Analyses (LS 701). Results are presented in Appendix B and described in the subsequent sections of this report.

4.0 Subsurface Conditions

The subsurface soil and groundwater conditions encountered in the test pits are presented on the attached Test Pit Logs in Appendix A. The stratigraphic boundaries indicated on the logs are inferred from observations during the field work and typically represent a transition from one soil type to another, sometime gradually. The boundaries should not be interpreted to represent exact planes of geologic change. The subsurface conditions have been confirmed in a series of widely spaced test locations and will vary between and beyond the test pit locations.

4.1 Stratigraphy

The subsurface conditions at the site generally consist of relatively shallow overburden deposits that vary in composition over bedrock. The deposits vary in composition across the site but is generally composed of topsoil overlying deposits of either cohesive/non-cohesive deposits and/or glacial till on bedrock. The upper portions of the subsurface deposits (likely as deep as 1 m across the entire site), have been disturbed and/or weathered either by previous land use or natural processes. The upper 1 m of overburden across the site should be considered disturbed.

4.1.1 Topsoil

Topsoil was encountered from the surface of the majority of test pit locations (23 of the test pits). The exact locations where topsoil was encountered are recorded on the respective Test Pit Logs.

Thickness of the topsoil ranges from 100 to 450 mm.

Assessments of organic matter content or other topsoil quality tests were beyond the scope of this study.

4.1.2 Non-cohesive Deposits

Non cohesive deposits were encountered either underlying the topsoil or from the surface at nine test pit locations (TP108-22, TP118-22, TP122-22, and TP127-22 through TP132-22).

The deposits vary in composition based on location but generally range from predominantly silty sand to sandy silt. Further details on deposit composition can be found on the respective Record of Test Pit Logs.

The thickness of these sand and silt deposit ranges from 0.2 to 1.0 m.

DPT results range from 1 to over 50, and generally increase with depth. It is estimated that the material ranges from very loose to very dense relative density.

Grain size distribution testing was completed on 2 samples of the non-cohesive deposits. The results are summarized on the Test Pit Logs and in Table 1 below. Detailed results diagrams are provided in Appendix B.

Table 1 Grain Size Distribution – Sands and Silts

Sample Location	Depth (mbgs)	Soil Description	% Gravel	% Sand	% Silt	% Clay
TP129-22 GS1	0.0 – 0.5	Sandy silt, trace clay, trace gravel	1	28	63	8
TP131-22 GS2	0.2 – 0.4	Silt sand, some clay	0	51	32	17

4.1.3 Cohesive Deposits

Cohesive deposits were encountered either underlying the topsoil or from the surface at three test pit locations (TP102-22, TP116-22, and TP117-22). The deposits are composed of brown clayey silt with trace sand with noted trace gravel in some locations. Organics were noted within the upper portions of the deposits. Further details on deposit composition can be found on the respective Record of Test Pit Logs.

The thickness of these clayey silt deposits range from 0.1 to 0.8 m.

DPT results range from 1 to 26, with low blow counts close to the surface. It is estimated that the material ranges from stiff to hard consistency.

Grain size distribution testing was completed on a sample of the clayey silt. The results are summarized on the Test Pit Logs and in Table 2 below. Detailed results diagrams are provided in Appendix B.

Table 2 Grain Size Distribution – Clayey Silt

Sample Location	Depth (mbgs)	Soil Description	% Gravel	% Sand	% Silt	% Clay
TP102-22 GS2	0.3 – 0.4	Clayey silt, trace sand	0	5	69	26

4.1.4 Glacial Till

Native deposits of glacial till were encountered across the site either underlying the topsoil, other deposits, or from the surface at eight test pit locations (TP103-22, TP116-22 through TP119-22, and TP124-22 through TP126-22). Glacial till deposits are heterogenous mixture of all grain sizes. The glacial till at this site is predominantly composed of sandy silt, with varying amounts of gravel and clay. A deposit of primarily silt and clay was noted in TP125-22 and silty or sandy gravel in TP124-22 and TP126-22. Cobbles and boulders were encountered within the glacial till deposits and are typical in these deposits.

The thickness of these glacial till deposits range from 0.3 to 0.9 m.

DPT results range from 9 to over 50, with low blow counts close to the surface. It is estimated that the material ranges from compact to very dense relative density.

Grain size distribution testing was completed on four samples of the glacial till. The results are summarized on the Test Pit Logs and in Table 3 below. Detailed results diagrams are provided in Appendix B.

Table 3 Grain Size Distribution – Glacial Till

Sample Location	Depth (mbgs)	Soil Description	% Gravel	% Sand	% Silt	% Clay
TP103-22 GS2	0.3 – 0.5	Sandy silt, some clay, some gravel	11	26	52	11
TP124-22 GS2	0.3 – 1.1	Silty gravel, some clay, trace sand	50	7	26	17
TP125-22 GS2	0.3 – 0.7	Silt and clay, some sand, trace gravel	1	11	46	42
TP126-22 GS2	0.3 – 1.1	Sandy gravel, some silt, trace clay	54	28	10	8

4.1.5 Bedrock

All test pits terminated with practical refusal on sound bedrock either from the surface or underlying the overburden. The depth and relative elevation to the top of the sound bedrock surface at each test pit location is summarized in Table 4.

Table 4 Depth to Top of Bedrock

Location	Depth to Top of Bedrock (mbgs)	Relative Elevation to Top of Bedrock (rel. El.)
TP101-22	0.4	102.3
TP102-22	0.4	103.9
TP103-22	0.5	107.2
TP104-22	Surface	111.7
TP105-22	Surface	112.1
TP106-22	Surface	112.7
TP107-22	Surface	111.7
TP108-22	0.5	110.3
TP109-22	0.1	111.5
TP110-22	0.1	111.1
TP111-22	0.1	110.1
TP112-22	Surface	110.2
TP113-22	0.2	109.7
TP114-22	0.1	108.9
TP115-22	0.1	108.9
TP116-22	1.1	102.1
TP117-22	1.4	102.6
TP118-22	1.2	101.8
TP119-22	1.1	100.9
TP120-22	0.2	102.6
TP121-22	0.3	101.9
TP122-22	0.4	99.9
TP123-22	0.2	100.0
TP124-22	1.1	102.0
TP125-22	0.7	104.3
TP126-22	1.1	105.7
TP127-22	0.6	105.3
TP128-22	1.0	105.4
TP129-22	0.5	103.7
TP130-22	0.4	102.1
TP131-22	0.4	101.6
TP132-22	0.8	104.9
TP133-22	0.2	102.1

Bedrock elevation across the site ranges in elevation from 99.9 to 112.7 rel. El. The bedrock grade appears to be higher in elevation from the northwest corner and drops towards the southeast corner of the site.

The portions of the bedrock are sufficiently weathered at test pit locations TP101-22, TP102-22, TP119-22, and TP132-22 to allow for removal of the weathered surface to depths ranging from 30 to 450 mm. The depths and elevations provided in the table above reference the top of sound bedrock, below the weathered zone.

Based on published Ontario Geological Survey maps, the site borders two rock formations. The underlying bedrock is anticipated to be composed of limestone and dolostone of the Gull River Formation or older Precambrian bedrock. Well records on site indicate that the upper 5 to 10 m is composed of limestone bedrock underlain by the older Precambrian rock, possibly granite.

Fissures were noted on the bedrock surface following excavation of the test pits at seven locations (TP104-22 through TP107-22, TP112-22, TP114-22, and TP115-22). The fissures measure approximately 0.2 m in width based on field observations and resemble possible karst topography. In particular the fissures noted in the areas of TP112-22, TP114-22, and TP115-22 extend at least 1.5 m deep.

4.2 Groundwater

Groundwater was not observed in any of the test pits during the excavation. Groundwater monitoring in site water supply wells was conducted as part of Cambium's Hydrogeological Investigation. The results of the monitoring are provided in a separate report. The groundwater levels in the 5 test wells installed on site range from 7.6 to 91.0 mbgs, within the bedrock. It is not anticipated that this water will be encountered during construction related activities.

Seasonal fluctuations and precipitation events may cause significant changes to the depth of the groundwater table over time.

5.0 Geotechnical Design Considerations

The following recommendations are based on test pit information and are intended to assist designers. Recommendations should not be construed as providing instructions to contractors, who should form their own opinions about site conditions. This report is based on the assumption that the design features relevant to the geotechnical analysis will be completed in accordance with applicable codes, standards, and guidelines of practice. It is possible that subsurface conditions beyond the test pit locations may vary from those observed. If significant variations are found before or during construction, Cambium should be contacted so that we can reassess our findings, if necessary.

It is intended for the existing property to be subdivided into 58 new residential lots, one commercial lot, and one block for stormwater management. It is assumed that single family low rise residential dwellings will be constructed on each of the lots with up to one basement level extending, at most 1.8 mbgs. The lots will be serviced with private groundwater well and septic systems. Associated residential roadways and driveways will be constructed as part of the development.

5.1 Excavations

All excavations must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). Excavations for the proposed development will extend through the topsoil, cohesive and non-cohesive deposits, glacial till, and possibly through the underlying bedrock, depending on final site grades.

The overburden materials at this site may be classified as Type 3 soils above the groundwater table in accordance with OHSA. Type 3 soils may be excavated with side slopes no steeper than 1H:1V.

Excavation 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 or the excavation sidewalls must be fully supported (shored).

The upper portions of the bedrock, if sufficiently weathered, could be removed mechanically with an excavator. Deeper excavation will require hoe-ramming, line drilling, or a combination of both.

Excavations made into bedrock can be cut vertically, provided that the rock faces are scaled and maintained to preclude the possibility of spalling. Where this is not possible, in areas where workers and/or equipment must enter the excavation, a protective mesh can be draped over the rock face. Alternatively, a trench box can be used in narrow excavation.

5.2 Groundwater Control

No groundwater seepage was noted during the course of this investigation. Groundwater flow through the underlying bedrock was observed as part of our hydrogeological investigation. The shallowest groundwater encountered in the hydrogeological investigation was observed at 7.6 mbgs within the bedrock.

It is highly unlikely that construction for the construction for the proposed dwellings on this site will advance significantly into the bedrock (deeper than 6 m). To account for seasonal fluctuations, provided that excavations for the proposed development do not advance deeper than 6 m into the underlying bedrock, groundwater flow into open excavations is not anticipated outside of major precipitation events.

For excavation work, it is expected that the groundwater inflow will be of limited extent. The groundwater may be allowed to drain into the excavation and then pumped out. In general, the volume of water anticipated to flow into the open excavations is such that temporary pumping from the excavation sump pumps is expected to suffice for the control of the groundwater.

5.3 Earthworks and Grade Raise

The final FFE of the proposed buildings are not yet known. Due to the relatively shallow bedrock depths, it is likely that the site grades will be raised in order to allow for the construction of any basement levels.

Any organic material, deleterious material, or refuse are not geotechnically suitable and must be removed from underlying any structures and the proposed pavement areas (roads, driveways, etc.).

Following excavation, the exposed subgrade must be inspected by geotechnical personnel prior to placing engineered fill for grade raise. Proof rolling of the bedrock is not considered necessary.

The site grades may then be raised using approved earth fill. All grade raises beneath structural elements such as building foundations or slabs should be raised using engineered fill such as that meeting Ontario Provincial Standards Specification (OPSS.MUNI) 1010 Granular B Type I. All grade raise fill must be placed at the site in loose lifts of 200 mm and compacted to a minimum of 98% of the Standard Proctor Maximum Dry Density (SPMDD) value and within 2 % of the optimum moisture content. Compaction requirements can be reduced to 90% in non settlement sensitive areas that do not support structural elements or hard surfaces (i.e. landscape areas). Any engineered fill must be placed under full time supervision and testing from geotechnical personnel.

It is estimated that some of the existing non-cohesive subsurface material may be re-used as grade raise fill. The fill material intended for reuse should be stockpiled and inspected by geotechnical personnel prior to re-use.

5.4 Foundation Design

It is anticipated that foundations for the proposed residential and commercial structures will be made on the underlying bedrock. The following foundation recommendations apply to both structure types at this site. Depth/Relative elevation to the top of sound bedrock is summarized on Table 1 of Section 4.1.5. Locations of each test pit is shown on Figure 1.

5.4.1 Frost Penetration

Based on climate data and design charts, the maximum frost penetration depth at the site is estimated at 1.7 m. Foundation footings should be provided with at least this amount of earth cover for frost protection purposes. If the required depth of earth cover is not practicable, a combination of earth cover and polystyrene insulation could be considered.

Foundations founded directly on clean, sound, unweathered bedrock do not require frost protection. The frost susceptibility of the bedrock subgrade below founding level should be assessed at the time of construction. If the subgrade is determined to be non-frost susceptible, the frost protection requirements could be waived.

5.4.2 Allowable Bearing Capacity

Foundations made to rest on top of sound bedrock can be designed using a maximum factored geotechnical resistance at **ULS** of **1000 kPa**. The SLS bearing is a function of acceptable settlement parameters. The settlement of foundations made on sound bedrock occurs as load is applied and is elastic, linear, and non-recoverable. Load tests have not been carried out by Cambium on the underlying bedrock, and as such, the net geotechnical reaction at SLS should be limited to **500 kPa** for spread footings.

Alternatively, where footings are founded above the top of sound bedrock, or where subexcavation is required below footings, foundation can be made to bear directly on a pad of Engineered Fill such as that conforming to OPSS.MUNI 1010 Granular B Type I. Any engineered fill placed below proposed foundations should be placed directly on sound bedrock. The imported engineered fill should be placed in maximum 200 mm thick lifts to at least 98 % of the SPMDD value. To allow for adequate spread of the loading below and beyond the footings, the engineered fill should extend a horizontal distance of at least 300 mm beyond the edge of the footings and then down and away from the edges at an angle of 1H:1V, or flatter. Excavations should be sized to accommodate fill placement. Foundations made on top of adequately compacted engineered fill should be sized using a net reaction at **SLS** of **150 kPa** and factored geotechnical resistance at **ULS** of **225 kPa**.

Settlement potential at the above-noted SLS loadings is less than 25 mm and differential settlement should be less than 20 mm between foundations supported on the engineered fill.

To reduce cracking in the footings and foundation walls where footings change between different subgrade materials, suitable transition zones should be created and the footings adequately reinforced.

Footings stepped from one level to another must be at a slope no exceeding 10H:7V from the outside edges of each foundation.

Alternatively, the proposed dwellings may be designed as slab on grade buildings in order to reduce bedrock excavation or grade raise requirements. Additional design commentary can be provided by Cambium if this design is chosen.

5.4.3 Possible Karst Topography

Based on published Ontario Geological Survey (OGS) maps, the property falls within the areas of inferred karst topography and potential karst topography. The local municipality may require a karst evaluation by qualified personnel as part of construction permitting.

Localized areas of possible karst topography were noted on the bedrock surface at select test pit locations. Fissures were observed on the surface of the bedrock in the areas of TP104-22 through TP107-22 and in the areas of TP112-22, TP114-22, and TP115-22. The fissures observed in TP112-22, TP114-22, and TP115-22 extend to at least 1.5 mbgs. The test pits across the remainder of the site showed no evidence of karst.

It is recommended, as part of construction and development, that the subgrades for the proposed buildings and septic systems in the areas where surficial fissuring was observed, are investigated for possible karst. These areas should include the lots located in the vicinity of the above noted test pits: Lots 1 through 6, 20, 26, 27, 33, 34, and 39 through 41.

Foundations for proposed buildings built over fissures may require additional reinforcement and/or the need for a woven geotextile reinforcement. Lean mix concrete may also be considered. The requirements should be assessed during the detailed design stage, or prior to construction, with additional investigations or an assessment of the subgrade by geotechnical personnel.

Septic systems can infiltrate the underlying groundwater depending on the extent of the surficial fissures and the installed supply well at this location. It is recommended that the groundwater wells in these areas are sealed within the underlying granite below the limestone. Alternatively, the extent of the fissures underlying septic beds can be thoroughly investigated to determine design restrictions for the septic systems and groundwater supply wells.

This investigation does not constitute a complete karst evaluation study.

5.5 Backfill and Compaction

The existing subsurface material may be reused as backfill material, if required, provided that it contains no organic content or deleterious materials and is free of boulders. Due to the potentially elevated silt content of the material, it is recommended that an adequate bond break is applied between the foundation wall and backfill to avoid frost adhesion.

Additionally, the on-site bedrock could be used as backfill, provided that it is crushed down to a grain size distribution generally meeting OPSS.MUNI 1010 Granular B Type I material requirements.

Where backfill will support areas of hard surfacing (pavements, walkways, etc.) the backfill should be placed in maximum 200 mm thick lifts and compacted to at least 95% of the SPMDD value. If compaction is not practicable due to the proximity for the foundation wall to the bedrock, 19 mm clear crushed stone may be used. The clear stone should be nominally compacted to a dense state and suitably wrapped with a nonwoven geotextile. Light, walk behind compaction equipment should be used in proximity to foundation walls.

Depending on the depth to bedrock underlying hard surfaced areas, frost tapers may be required in order to reduce the effects of differential frost heaving. If fill material or native material is encountered within 1.7 m from the surface underlying paved areas and where these hard surfaced areas abut the proposed structures, it is suggested that frost tapers be constructed. The frost tapers should be constructed at a 1 horizontal to 1 vertical slope, or flatter.

5.5.1 Lateral Resistance

The appropriate values for use in the design of structures subject to unbalanced earth pressures at this site are tabulated as follows in Table 5:

Table 5 Earth Pressure Design Values

Stratum/Parameter	γ	φ	Ka	Ko	Kp
Earth Fill	19	30	0.33	0.50	3.00
Granular Backfill	22	35	0.27	0.42	3.70

Where: γ = bulk unit weight of soil (kN/m^3)

φ = internal angle of friction (degrees)

K_a = Rankine active earth pressure coefficient (dimensionless)

K_o = Rankine at-rest earth pressure coefficient (dimensionless)

K_p = Rankine passive earth pressure coefficient (dimensionless)

The above earth pressure parameters pertain to a horizontal grade condition behind a retaining structure. Values of earth pressure parameters for an inclined retained grade condition will vary.

Active pressure is applied on the backfilled side of the wall towards the side with lower pressure (overturning force). At rest lateral earth pressures are considered for the retained structures that are braced and experience no movement. For example basement walls, that are internally braced by the structure, should be considered at rest. Unbraced retaining walls are typically subject to active pressures for design. The design of retaining structures will require input from the respective structural engineer.

Walls subject to unbalanced earth pressures must be designed to resist a pressure that can be calculated based on the following equation:

$$P = K[\gamma(h - h_w) + \gamma' h_w + q] + \gamma_w h_w$$

Where,	P	=	the horizontal pressure at depth, h (m)
	K	=	the earth pressure coefficient
	hw	=	the depth below the ground water level (m)
	γ	=	the bulk unit weight of soil, (kN/m3)
	γ'	=	the submerged unit weight of the exterior soil, (γ - 9.8 kN/m3)
	q	=	the complete surcharge loading (kPa)

The wall backfill must be drained effectively to eliminate hydrostatic pressures on the wall that would otherwise act in conjunction with the earth pressure. In this case, the above equation is simplified to:

$$P = K[\gamma h + q]$$

5.6 Sliding Resistance

The factored geotechnical resistance to sliding of foundation elements is developed by friction between the base of the concrete footing and the soil or bedrock. This friction (**R**) depends on the normal load at the soil contact (**N**) and the frictional resistance of the soil (**tan φ**) expressed as $R_f = N\ tan\varphi$, which is the unfactored resistance. The factored geotechnical resistance at ULS is $R_f = 0.8\ N\ tan\varphi$ for foundations on soil, and $R_f = 0.4\ N\ tan\varphi$ for foundations on unweathered bedrock.

The internal friction angle (φ) for engineered fill is provided in Table 5. A value of **40**° can be used for the φ for unweathered sound bedrock.

5.7 Perimeter Drainage and Basement Floor Slab Drainage

Groundwater seepage was not observed during this investigation.

Perimeter foundation drainage consisting of geotextile wrapped perforated 100 mm diameter pipes surrounded by a trench of 19 mm clear stone should be provided around the foundations.

For basement floor slabs, a 200 mm thick layer of 19 mm diameter clear crushed stone should be placed and connected to perimeter drains below the floor slab to assist with drainage.

The drainage system should outlet to a suitable discharge point under gravity flow away from the structures, or to a sump pit. The design of the system must conform to applicable plumbing code requirements.

Additional underfloor drainage may be required depending on final design details.

5.8 Commercial Block Slab-on-Grade Construction and Drainage

It is understood that one of the subdivided lots will be reserved for a commercial development and it is anticipated that this development will consist of slab-on-grade design (i.e. no basement).

All organic material and deleterious material must be removed prior to constructing the slab on grade. These materials do not constitute an adequate subgrade for support of a slab on grade. Compacted engineered fill such as material meeting OPSS.MUNI 1010 Granular A, or B Type I or II built on the sound bedrock is suitable for the support of a conventional slab on grade. The subgrade must be approved by geotechnical personnel. The existing subsurface materials and topsoil at this site should not be reused as fill under the slab. Any underlying bedrock should be free of all organic material.

The modulus of subgrade reaction appropriate for slab on grade design at the site is as follows:

Engineered Fill: 22,000 kPa/m

The subgrade for the slab must be cut-neat and inspected by geotechnical personnel, prior to the placement of an aggregate base. Proof rolling exposed bedrock is not considered necessary. If there are areas containing excessive amounts of deleterious/organic material or moisture, they must be locally sub-excavated and backfilled with Engineered Fill such as OPSS Granular B (Type I or II) and compacted to a minimum of 98% Standard Proctor Maximum Dry Density (SPMDD). The on-site bedrock could be used as grade raise fill provided that it is crushed down to a grain size distribution generally meeting OPSS.MUNI 1010 Granular B Type I material requirements.

It is recommended that the slab be provided with a capillary moisture barrier. This is made by placing the slab on a minimum 200 mm layer of clear stone and nominally compacted by vibration to a dense state. Alternatively, the capillary moisture barrier can be composed of a 200 mm thick layer of OPSS.MUNI 1010 Granular A, compacted to a minimum 98% of the SPMDD. Underslab drainage is not required beyond the capillary moisture barrier provided the floor slab elevation is set at 300 mm or higher than the exterior grade.

Perimeter foundation drainage is not considered necessary for slab-on-grade structures provided that the finished floor slab elevation is set 300 mm or higher than the exterior grade.

5.9 Seismic Site Classification

The Ontario Building Code (OBC) specifies that the structures should be designed to withstand forces due to earthquakes. For the purpose of earthquake design, geotechnical information shall be used to determine the "Site Class".

It is anticipated that the foundations of the proposed commercial structure will be underlain by sound bedrock of the Gull River Formation.

Based on our experience with foundations supported on this formation and in accordance with Table 4.1.8.4.A of the OBC (2006), it is recommended that Site Class "B" be applied for structural design at the Site.

The above site class designation assumes that sound bedrock will be located within 1m of the base of the foundations. If foundations are supported on compacted engineered fill extending deeper than 1 m from under the foundations, the seismic site class will have to be reduced to accurately reflect the site conditions.

5.10 Pavement Design Consideration

5.10.1 Subgrade Preparation

The performance of the pavement is dependent upon proper subgrade preparation. All topsoil and organic materials are to be removed from the subgrade. The subgrade should be proof rolled and inspected by geotechnical personnel. Any areas where rutting or appreciable deflection is noted should be sub-excavated and replaced with suitable earth fill. The earth fill may be taken from other parts of the site for reuse, provided that the material is free of organics. The fill should be compacted to at least 98% of SPMDD. Subgrades composed of exposed bedrock do not require proofrolling.

The most severe loading conditions on pavement subgrades may occur during construction, and subgrades may become disturbed due to construction operations. Therefore the recommended pavement structure provided may not be adequate due to the presence of localized disturbed areas and it may be necessary to increase the thickness of the Granular B Type II subbase and/or incorporate a woven geotextile separator between the subgrade surface and the granular base. The requirement for an increase in the pavement structure and/or incorporating geotextile will be evaluated by geotechnical personnel during proof roll inspections.

5.10.2 Flexible Pavement Structure

The pavement structure recommended in Table 6 below assumes that traffic flow and access will be limited to residential use and that the subgrades will be prepared as described above. Light duty pavement can be considered for parking areas frequented exclusively by passenger vehicles. Heavy duty pavement structure should be considered for rural roads used by occasional emergency and service vehicles.

Table 6 Recommended Minimum Pavement Structure

Pavement Layer	Light Duty	Heavy Duty
Surface Course Asphalt	50 mm HL3 or HL4	40 mm HL3 or HL4
Binder Course Asphalt	-	50 mm HL8
Granular Base	150 mm OPSS 1010 Granular A	150 mm OPSS 1010 Granular A
Granular Subbase	250 mm OPSS 1010 Granular B Type I or II	300 mm OPSS 1010 Granular B Type I or II

The thickness of the subbase layer can be reduced up to 0 mm where shallow bedrock is encountered within the pavement thickness depth.

Material and thickness substitutions must be approved by the Design Engineer. The thickness of the subbase layer could also be increased at the discretion of the Engineer, to accommodate site conditions at the time of construction, including soft or weak subgrade soil replacement.

Compaction of the subgrade should be verified by the Engineer prior to placing the granular fill. Granular layers should be placed in no more than 300 mm thick lifts and compacted to at least 98% of SPMDD (ASTM D698) standard. The granular materials specified should conform to OPSS standards, as confirmed by appropriate materials testing.

5.10.3 Pavement Transitions

Existing asphaltic concrete should be neatly saw cut at pavement transition areas. The joints should be tack coated in accordance with OPSS.MUNI 310 requirements. In order to avoid differential frost heaving where granular thicknesses vary between different pavements, a gradual frost taper should be provided.

5.10.4 Pavement Drainage

The design of a storm water management system is beyond the scope of this investigation; however it is recommended that the subgrade, subbase, base, and asphalt surfaces should be shaped and crowned to promote drainage of the pavement structure.

If the roads will have an urban cross-section, subdrains can be considered along the curblines adjacent to the subbase of the roadway structure to aid in stormwater drainage.

For rural roadways with swales or ditches, the granular base and subbase materials should extend horizontally to the ditches/swales. Where possible, the bottom of the swales/ditches should be at least about 0.3 metres below the bottom of the subbase layer.

5.11 Storm Water Management Pond

It is understood that a storm water management block is proposed on the eastern side of the development within a location of an already existing pond (near TP125-22, TP128-22, and TP129-22). The depth and design details of what will be constructed were not available at the time of writing this report. It is recommended that Cambium review the preliminary design drawings in order to confirm the relevant geotechnical considerations prior to finalizing the design.

The area of the proposed pond encountered bedrock at depths ranging from 0.5 to 1 mbgs. All excavation work should be carried out as described in Sections 5.1 and 5.2. It is recommended that the overburden material along the banks of the proposed pond be sloped at an angle of 3H:1V or flatter. Slope banks composed of exposed bedrock can be sloped at an angle of 1:1V, or flatter. The recommended slopes can be applied to wet or dry ponds.

6.0 Report Limitations

6.1 Design Review and Inspections

Testing and inspections should be carried out during construction operations to test concrete and to examine and approve subgrade conditions, placement and compaction of fill materials, granular base courses, and asphaltic concrete.

Cambium should be contacted to review and approve design drawings, prior to tendering or commencing construction, to ensure that all pertinent geotechnical-related factors have been addressed. It is important that onsite geotechnical supervision be provided at this site for excavation and backfill procedures, deleterious soil removal, subgrade inspections and compaction testing.

6.2 Changes in Site and Additional Investigations

Subsurface conditions can be altered by the passage of sufficient time, natural occurrences, and human intervention. In particular, consideration should be given to contractual responsibilities as they relate to control of groundwater seepage, disturbance of soils, and frost protection.

This geotechnical engineering report is intended for preliminary planning and design purposes only. Detailed design of the proposed development has not been completed. The recommendations and the engineering advice offered in this report should be reviewed when additional design details are known. Additional investigation, including boreholes, may be required.

7.0 Closing

Please note that this work program and report are governed by the attached Qualifications and Limitations. If you have questions or comments regarding this document, please do not hesitate to contact the undersigned at (705) 742-7900.

Respectfully submitted,

Cambium Inc.

DocuSigned by:
0B68D45279A94B7...

Stuart Baird, M.Eng., P.Eng.
Director of Technical Operations

DS

2025-12-03

DocuSigned by:
6439A79ECBB1496...

Blasco Vijayabaskaran, P.Eng.
Geotechnical Engineer

bv/SB

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8.0 Standard Limitations

Limited Warranty

In performing work on behalf of a client, Cambium relies on its client to provide instructions on the scope of its retainer and, on that basis, Cambium determines the precise nature of the work to be performed. Cambium undertakes all work in accordance with applicable accepted industry practices and standards. Unless required under local laws, other than as expressly stated herein, no other warranties or conditions, either expressed or implied, are made regarding the services, work or reports provided.

Reliance on Materials and Information

The findings and results presented in reports prepared by Cambium are based on the materials and information provided by the client to Cambium and on the facts, conditions and circumstances encountered by Cambium during the performance of the work requested by the client. In formulating its findings and results into a report, Cambium assumes that the information and materials provided by the client or obtained by Cambium from the client or otherwise are factual, accurate and represent a true depiction of the circumstances that exist. Cambium relies on its client to inform Cambium if there are changes to any such information and materials. Cambium does not review, analyze or attempt to verify the accuracy or completeness of the information or materials provided, or circumstances encountered, other than in accordance with applicable accepted industry practice. Cambium will not be responsible for matters arising from incomplete, incorrect or misleading information or from facts or circumstances that are not fully disclosed to or that are concealed from Cambium during the provision of services, work or reports.

Facts, conditions, information and circumstances may vary with time and locations and Cambium's work is based on a review of such matters as they existed at the particular time and location indicated in its reports. No assurance is made by Cambium that the facts, conditions, information, circumstances or any underlying assumptions made by Cambium in connection with the work performed will not change after the work is completed and a report is submitted. If any such changes occur or additional information is obtained, Cambium should be advised and requested to consider if the changes or additional information affect its findings or results.

When preparing reports, Cambium considers applicable legislation, regulations, governmental guidelines and policies to the extent they are within its knowledge, but Cambium is not qualified to advise with respect to legal matters. The presentation of information regarding applicable legislation, regulations, governmental guidelines and policies is for information only and is not intended to and should not be interpreted as constituting a legal opinion concerning the work completed or conditions outlined in a report. All legal matters should be reviewed and considered by an appropriately qualified legal practitioner.

Site Assessments

A site assessment is created using data and information collected during the investigation of a site and based on conditions encountered at the time and particular locations at which fieldwork is conducted. The information, sample results and data collected represent the conditions only at the specific times at which and at those specific locations from which the information, samples and data were obtained and the information, sample results and data may vary at other locations and times. To the extent that Cambium's work or report considers any locations or times other than those from which information, sample results and data was specifically received, the work or report is based on a reasonable extrapolation from such information, sample results and data but the actual conditions encountered may vary from those extrapolations.

Only conditions at the site and locations chosen for study by the client are evaluated; no adjacent or other properties are evaluated unless specifically requested by the client. Any physical or other aspects of the site chosen for study by the client, or any other matter not specifically addressed in a report prepared by Cambium, are beyond the scope of the work performed by Cambium and such matters have not been investigated or addressed.

Reliance

Cambium's services, work and reports may be relied on by the client and its corporate directors and officers, employees, and professional advisors. Cambium is not responsible for the use of its work or reports by any other party, or for the reliance on, or for any decision which is made by any party using the services or work performed by or a report prepared by Cambium without Cambium's express written consent. Any party that relies on services or work performed by Cambium or a report prepared by Cambium without Cambium's express written consent, does so at its own risk. No report of Cambium may be disclosed or referred to in any public document without Cambium's express prior written consent. Cambium specifically disclaims any liability or responsibility to any such party for any loss, damage, expense, fine, penalty or other such thing which may arise or result from the use of any information, recommendation or other matter arising from the services, work or reports provided by Cambium.

Limitation of Liability

Potential liability to the client arising out of the report is limited to the amount of Cambium's professional liability insurance coverage. Cambium shall only be liable for direct damages to the extent caused by Cambium's negligence and/or breach of contract. Cambium shall not be liable for consequential damages.

Personal Liability

The client expressly agrees that Cambium employees shall have no personal liability to the client with respect to a claim, whether in contract, tort and/or other cause of action in law. Furthermore, the client agrees that it will bring no proceedings nor take any action in any court of law against Cambium employees in their personal capacity.

Appended Figures

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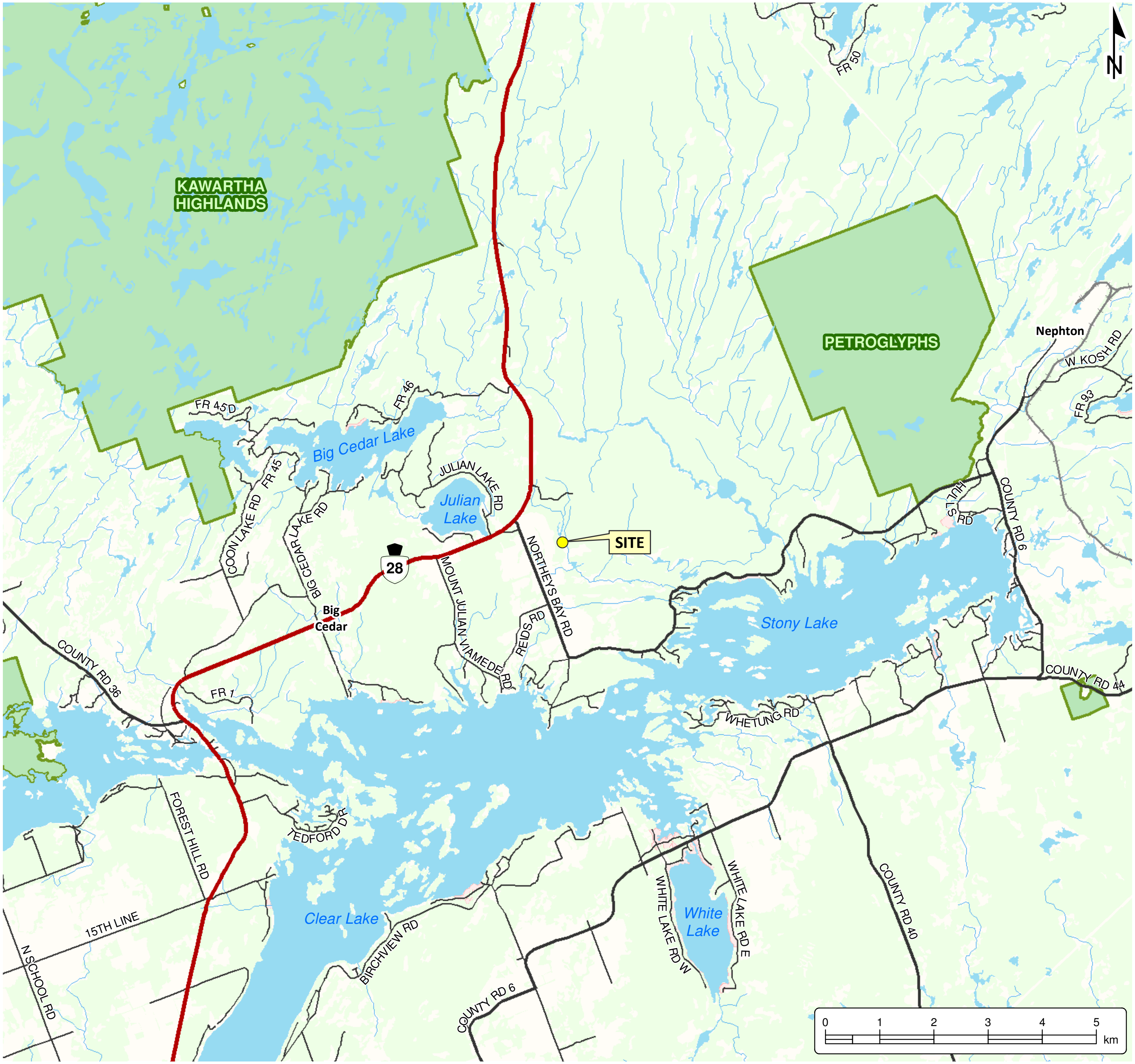

GEOTECHNICAL INVESTIGATION

ERIC CHALLENGER
65 Northeys Bay Road,
Woodview, Ontario

LEGEND

- Highway
- Major Road
- Minor Road
- Railroad
- Watercourse
- Water Area
- Provincial Park
- Wooded Area
- Built Up Area

Notes:
- Base mapping features are © Queen's Printer of Ontario, 2019 (this does not constitute an endorsement by the Ministry of Natural Resources or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

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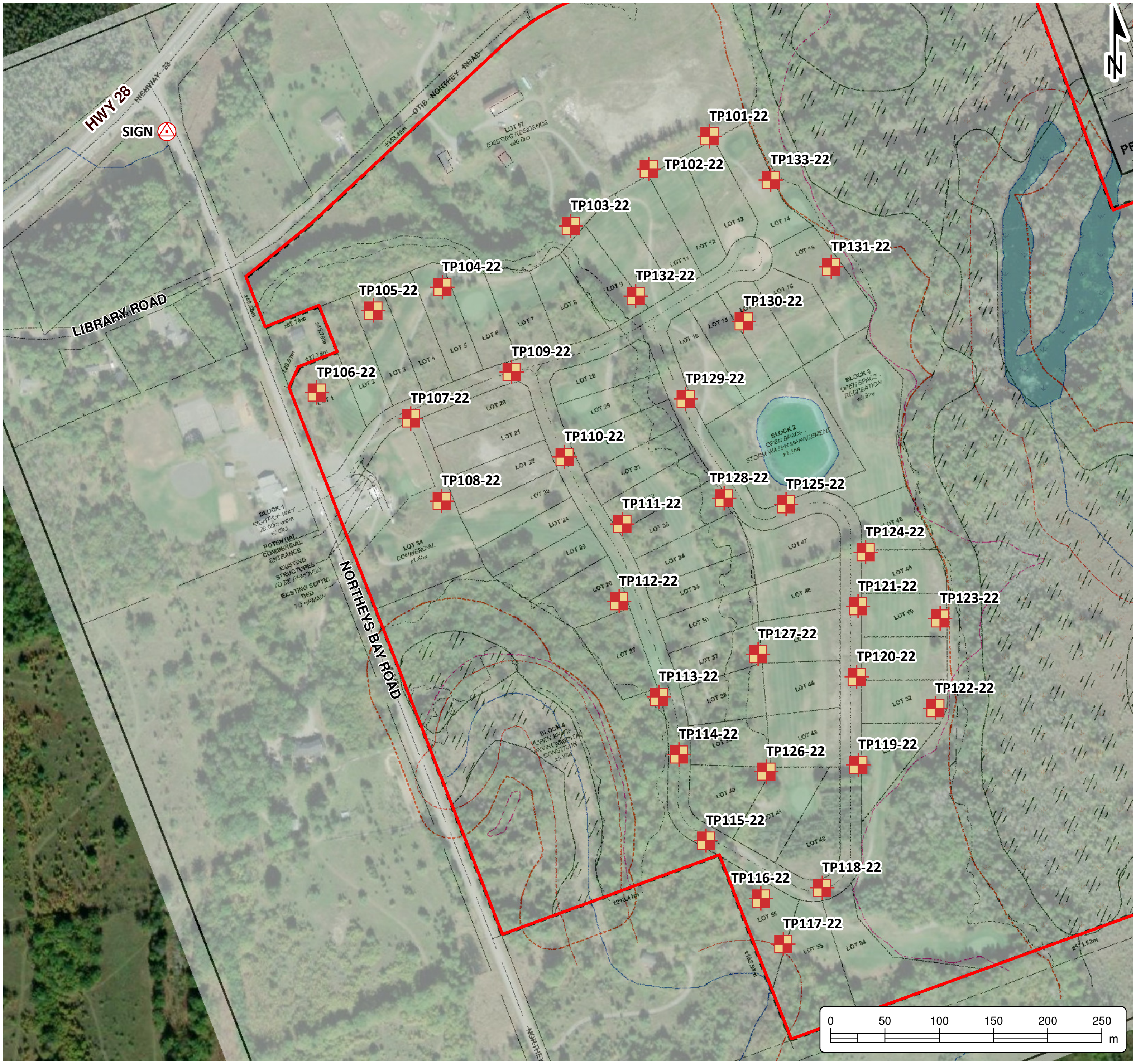
SITE LOCATION MAP

Project No.: 15101-001	Date: November 2022 Rev.:
Scale: 1:100,000	Projection: NAD 1983 UTM Zone 17N

Created by: SJV	Checked by: SNR	Figure: 1

O:\GIS\MXDs\15100-15199\15101-001 Woodview Golf - Eric Challenger - Hydrog & Geo - Woodview Golf Subdivision\2022-11-15 FIG 1 - Site Location Plan.mxd

Docusign Envelope ID: FE9B494C-1543-4EB3-8222-F0F96BF5362E



GEOTECHNICAL INVESTIGATION

ERIC CHALLENGER
65 Northeys Bay Road,
Woodview, Ontar

LEGEND

Type

Benchmark

Test Pit

Site (approximate)

Notes:
- Benchmark is the steel foundation stick-up of the road sign for Highway 56 (shown on map labelled "SIGN"), with an assumed elevation of 100.0 m.
- Base mapping features are © Queen's Printer of Ontario, 2019 (this does not constitute an endorsement by the Ministry of Natural Resources or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

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TEST PIT LOCATION PLAN

Project No.: 15101-001		Date: November 2022	Rev.:
Scale: 1:5,000		Projection: NAD 1983 UTM Zone 17N	
Created by: SJV	Checked by: SNR	Figure: 2	

O:\GIS\MXDs\15100-15199\15101-001 Woodview Golf - Eric Challenger - Hydrog & Geo - Woodview Golf Subdivision\2022-11-15 FIG 2 - Test Pit Location Plan.mxd

Appendix A

Test Pit Logs

TABLE 1: TEST PIT LOGS
65 Northey's Bay Road, Township of North Kawartha - Test Pit Investigation
Technician: James Goodwin
Cambium Reference No. 15101-001
Completed: November 4, 2022

Test Pit ID	Depth (mbgs[1])	Soil Sample	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP101-22 17T 728027.6 m E, 4941799.3 m N 102.69 m rel	0.0 - 0.30 0.30 - 0.40	GS1 GS2	TOPSOIL: 300 mm Grey weathered/fractured bedrock, interbedded with silty layers, moist **Test pit terminated at 0.40 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45	2 4 50/100
TP102-22 17T 727971.8 m E, 4941769.0 m N 104.36 m rel	0.0 - 0.30 0.30 - 0.40 0.40 - 0.43	GS1 GS2	TOPSOIL: 300 mm CLAYEY SILT: Dark brown, clayey silt, trace sand, est. firm to stiff, moist (GSA: 0% Gravel, 5% Sand, 69% Silt, 26% Clay) Grey weathered/fractured bedrock, interbedded with silty layers, moist **Test pit terminated at 0.43 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45	6 6 50/50
TP103-22 17T 727899.9 m E, 4941716.4 m N 107.69 m rel	0.0 - 0.25 0.25 - 0.50	GS1 GS2	TOPSOIL: 250 mm GLACIAL TILL: Light brown sandy silt, some gravel, some clay, est. compact, moist (GSA: 11% Gravel, 26% Sand, 52% Silt, 11% Clay) **Test pit terminated at 0.50 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45	6 10 50/75
TP104-22 17T 727781.4 m E, 4941659.7 m N 111.70 m rel	0.0		**Bedrock was encountered at ground surface** -Fissures of up to 0.15 m in width were encountered in the bedrock, resembling karst		N/A
TP105-22 17T 727717.8 m E, 4941638.0 m N 112.14 m rel	0.0		**Bedrock was encountered at ground surface** -Fissures of up to 0.15 m in width were encountered in the bedrock, resembling karst		N/A
TP106-22 17T 727665.7 m E, 4941562.2 m N 112.71 m rel	0.0		**Bedrock was encountered at ground surface** -Fissures of up to 0.15 m in width were encountered in the bedrock, resembling karst		N/A
TP107-22 17T 727752.3 m E, 4941538.4 m N 111.70 m rel	0.0		**Bedrock was encountered at ground surface** -Fissures of up to 0.15 m in width were encountered in the bedrock, resembling karst		N/A
TP108-22 17T 727780.8 m E, 4941462.0 m N 110.73 m rel	0.0 - 0.45	GS1	SILTY SAND: Brown, silty sand, est. compact to dense, moist -organics encountered to 0.10 mbgs **Test pit terminated at 0.45 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60	3 14 29 50/25
TP109-22 17T 727845.4 m E, 4941581.2 m N 111.60 m rel	0.0 - 0.10	GS1	TOPSOIL: 100 mm **Test pit terminated at 0.10 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP110-22 17T 727894.1 m E, 4941502.8 m N 111.16 m rel	0.0 - 0.10	GS1	TOPSOIL: 100 mm **Test pit terminated at 0.10 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP111-22 17T 727947.3 m E, 4941440.6 m N 110.15 m rel	0.0 - 0.10	GS1	TOPSOIL: 100 mm **Test pit terminated at 0.10 mbgs on bedrock** Test pit open and dry upon completion		N/A

1. mbgs = metres below ground surface
2. Dynamic probe penetration test, consisting of driving a 19 mm diameter steel rod 150 mm into the soil with an 8 kg hammer falling 750 mm.

TABLE 1: TEST PIT LOGS
65 Northey's Bay Road, Township of North Kawartha - Test Pit Investigation
Technician: James Goodwin
Cambium Reference No. 15101-001
Completed: November 4, 2022

Test Pit ID	Depth (mbgs[1])	Soil Sample	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
Test Pit ID	Depth (mbgs[1])	Soil Sample	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP112-22 17T 727944.4 m E, 4941369.3 m N 110.24 m rel	0.0		**Bedrock was encountered at ground surface** -Fissures of up to 0.20 m in width and at least 1.5 m deep were encountered in the bedrock, resembling karst		N/A
TP113-22 17T 727981.4 m E, 4941281.1 m N 109.90 m rel	0.0 - 0.20	GS1	TOPSOIL: 200 mm **Test pit terminated at 0.20 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP114-22 17T 727999.6 m E, 4941227.7 m N 109.01 m rel	0.0 - 0.10	GS1	TOPSOIL: 100 mm **Test pit terminated at 0.10 mbgs on bedrock** -Fissures of up to 0.15 m in width and at least 1.5 m deep were encountered in the bedrock, resembling karst Test pit open and dry upon completion		N/A
TP115-22 17T 728024.4 m E, 4941147.9 m N 108.97 m rel	0.0 - 0.10	GS1	TOPSOIL: 100 mm **Test pit terminated at 0.10 mbgs on bedrock** -Fissures of up to 0.15 m in width and at least 1.5 m deep were encountered in the bedrock, resembling karst Test pit open and dry upon completion		N/A
TP116-22 17T 728075.2 m E, 4941094.4 m N 103.21 m rel	0.0 - 0.75 0.75 - 1.10	GS1	CLAYEY SILT: Brown, clayey silt, trace sand, est. firm to very stiff, moist -organics encountered from surface to 0.15 mbgs GLACIAL TILL: Brown silty sand, some gravel, trace clay, est. loose to very dense, moist **Test pit terminated at 1.10 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75 0.75 - 0.90 0.90 - 1.05	1 3 6 9 8 9 50/25
TP117-22 17 T 728095.7 m E, 4941052.3 m N 103.90 m rel	0.0 - 0.30 0.30 - 0.75 0.75 - 1.35	GS1 GS2 GS3	TOPSOIL: 300 mm CLAYEY SILT: Brown, clayey silt, trace sand, trace gravel, est. stiff to very hard, moist GLACIAL TILL: Brown, silty sand, some gravel, trace clay, est. compact to dense, moist -cobbles and boulders encountered throughout **Test pit terminated at 1.35 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75 1.20 - 1.35 1.35 - 1.50	3 9 26 23 8 32 50/25
TP118-22 17T 728131.7 m E, 4941104.3 m N 102.97 m rel	0.0 - 0.45 0.45 - 0.90 0.90 - 1.20	GS1 GS2	TOPSOIL: 450 mm SANDY SILT: Dark brown, sandy silt, trace clay, with organics, compact, moist GLACIAL TILL: brown, sandy silt, trace clay, trace gravel, with cobbles and boulders, est. dense, moist **Test pit terminated at 1.20 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75 0.75 - 0.90	1 3 7 20 48 50
TP119-22 17 T 728165.1 m E, 4941218.3 m N 102.00 m rel	0.0 - 0.40 0.40 - 1.00 1.00 - 1.10	GS1 GS2 GS3	TOPSOIL: 400 mm GLACIAL TILL: Brown, sandy silt, some clay, weathered bedrock/shale throughout, moist Grey weathered/fractured bedrock, interbedded with silty layers, moist **Test pit terminated at 1.10 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75	3 6 12 30 50/75
TP120-22 17T 728163.6 m E, 4941299.4 m N 102.70 m rel	0.0 - 0.15	GS1	TOPSOIL: 150 mm **Test pit terminated at 0.15 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP121-22 17T 4941364.7 m E, 4920940.9 m N 102.18 m rel	0.0 - 0.25	GS1	TOPSOIL: 250 mm **Test pit terminated at 0.25 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP122-22 17 T 728235.9 m E, 4941270.6 m N 100.27 m rel	0.0 - 0.25 0.25 - 0.40	GS1 GS2	TOPSOIL: 250 mm SANDY SILT: Brown, sandy silt, trace clay, trace gravel, est. compact, moist **Test pit terminated at 0.40 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP123-22 17T 728240.3 m E, 4941353.5 m N 100.17 m rel	0.0 - 0.20	GS1	TOPSOIL: 200 mm **Test pit terminated at 0.20 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP124-22 17T 728171.7 m E, 4941414.6 m N 103.09 m rel	0.0 - 0.25 0.25 - 1.10	GS1 GS2	TOPSOIL: 250 mm GLACIAL TILL: Brown, silty gravel, some clay, trace sand, est. compact to dense, moist (GSA: 50% Gravel, 7% Sand, 26% Silt, 17% Clay) **Test pit terminated at 1.10 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75 0.75 - 0.90 0.90 - 1.05	1 5 10 18 24 35 50/100

1. mbgs = metres below ground surface
2. Dynamic probe penetration test, consisting of driving a 19 mm diameter steel rod 150 mm into the soil with an 8 kg hammer falling 750 mm.

TABLE 1: TEST PIT LOGS
65 Northey's Bay Road, Township of North Kawartha - Test Pit Investigation
Technician: James Goodwin
Cambium Reference No. 15101-001
Completed: November 4, 2022

Test Pit ID	Depth (mbgs[1])	Soil Sample	Material Description	Depth (m)	DPT[2] (Blows/150 mm)
TP125-22 17T 728098.3 m E, 4941459.0 m N 104.94 m rel	0.0 - 0.25 0.25 - 0.65	GS1 GS2	TOPSOIL: 250 mm GLACIAL TILL: Brown, silt and clay, some sand, trace gravel, est. very stiff to hard, moist (GSA: 1% Gravel, 11% Sand, 46% Silt, 42% Clay) **Test pit terminated at 0.65 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60	1 5 30 50/125
TP126-22 17T 728080.2 m E, 4941211.9 m N 106.76 m rel	0.0 - 0.25 0.25 - 1.10	GS1 GS2	TOPSOIL: 250 mm GLACIAL TILL: Brown sandy gravel, some silt, trace clay, est. compact to dense, moist to wet (GSA: 54% Gravel, 28% Sand, 10% Silt, 8% Clay) -cobbles and boulders encountered throughout **Test pit terminated at 1.10 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75 0.75 - 0.90	1 6 10 23 24 35
TP127-22 17T 728072.6 m E, 4941320.4 m N 105.94 m rel	0.0 - 0.25 0.25 - 0.60	GS1 GS2	TOPSOIL: 250 mm SANDY SILT: Brown, sandy silt, trace clay, est. compact, moist **Test pit terminated at 0.60 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60	1 5 8 50/125
TP128-22 17T 728041.2 m E, 4941464.4 m N 106.35 m rel	0.0 - 0.95	GS1	SANDY SILT: Brown sandy silt, some clay, trace gravel, est. compact, moist -organics encountered from surface to 0.30 mbgs -cobbles encountered from 0.60 mbgs to 0.85 mbgs **Test pit terminated at 0.95 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60	1 5 10 50/100
TP129-22 17T 728005.8 m E, 4941556.3 m N 104.10 m rel	0.0 - 0.45	GS1	SANDY SILT: Brown, sandy silt, trace clay, trace gravel, est. compact to dense, moist to wet (GSA: 1% Gravel, 28% Sand, 63% Silt, 8% Clay) -organics encountered from surface to 0.15 mbgs **Test pit terminated at 0.45 mbgs on bedrock** Test pit open with minimal water pooling at the base of the test pit upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45	1 5 50
TP130-22 17T 728059.1 m E, 4941628.0 m N 102.46 m rel	0.0 - 0.20 0.20 - 0.40	GS1 GS2	TOPSOIL: 200 mm SANDY SILT: Light brown, sandy silt, some clay, trace gravel, est. compact, moist to wet **Test pit terminated at 0.40 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP131-22 17T 728140.1 m E, 4941678.4 m N 101.97 m rel	0.0 - 0.15 0.20 - 0.40	GS1 GS2	TOPSOIL: 150 mm SILTY SAND: Brown, silty sand, some clay, compact, moist to wet (GSA: 0% Gravel, 51% Sand, 32% Silt, 17% Clay) **Test pit terminated at 0.40 mbgs on bedrock** Test pit open and dry upon completion		N/A
TP132-22 17 T 728084.3 m E, 4941758.9 m N 105.63 m rel	0.0 0.30 0.30 - 0.75	GS1 GS2	SILTY SAND: Brown silty sand, some clay, est. loose to compact, moist Grey weathered/fractured bedrock, interbedded with silt, some sand and trace clay, moist to wet **Test pit terminated at 0.75 mbgs on bedrock** Test pit open and dry upon completion	0.0 - 0.15 0.15 - 0.30 0.30 - 0.45 0.45 - 0.60 0.60 - 0.75	1 4 10 37 50/100
TP133-22 17T 728084.3 m E, 4941758.9 m N 102.26 m rel	0.0 - 0.20	GS1	TOPSOIL: 200 mm **Test pit terminated at 0.20 mbgs on bedrock** Test pit open and dry upon completion		N/A

1. mbgs = metres below ground surface
2. Dynamic probe penetration test, consisting of driving a 19 mm diameter steel rod 150 mm into the soil with an 8 kg hammer falling 750 mm.

Appendix B

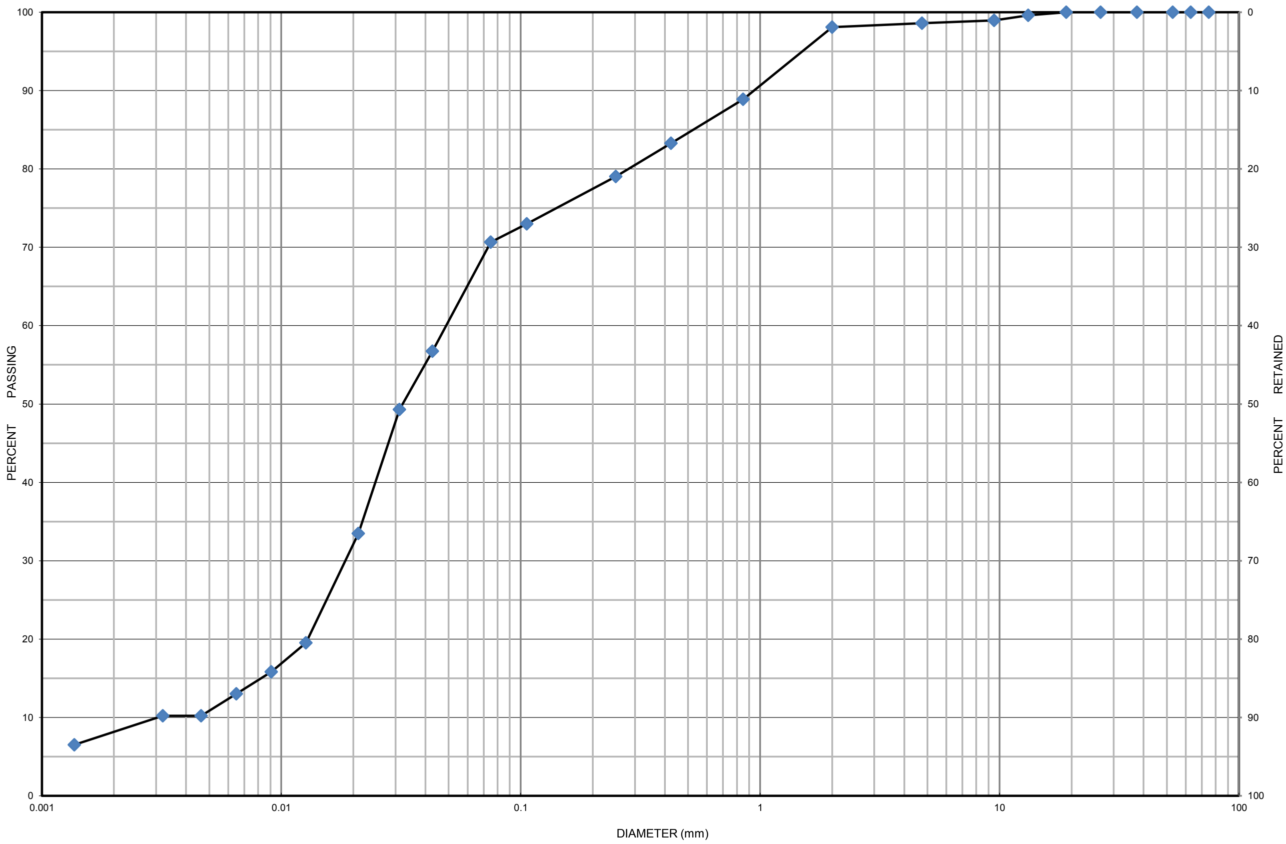
Soil Laboratory Testing Results

Docusign Envelope ID: FE9B494C-1543-4EB3-8222-F0F96BF5362E

Grain Size Distribution Chart

Project Number:	*15101-001*	**Client:**	*Woodview Golf - Eric Challenger*		
Project Name:	*Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*				
Sample Date:	*November 4, 2022*	**Sampled By:**	*James Goodwin - Cambium Inc.*		
Location:	*TP 129-22 GS 1*	**Depth:**	*0 m to 0.5 m*	**Lab Sample No:**	*S-22-1683*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 129-22	GS 1	0 m to 0.5 m	1	28	63	8	9.3

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sandy Silt trace Clay trace Gravel	ML	0.049	0.018	0.003	16.33	2.20

Additional information available upon request

Issued By: (Senior Project Manager)

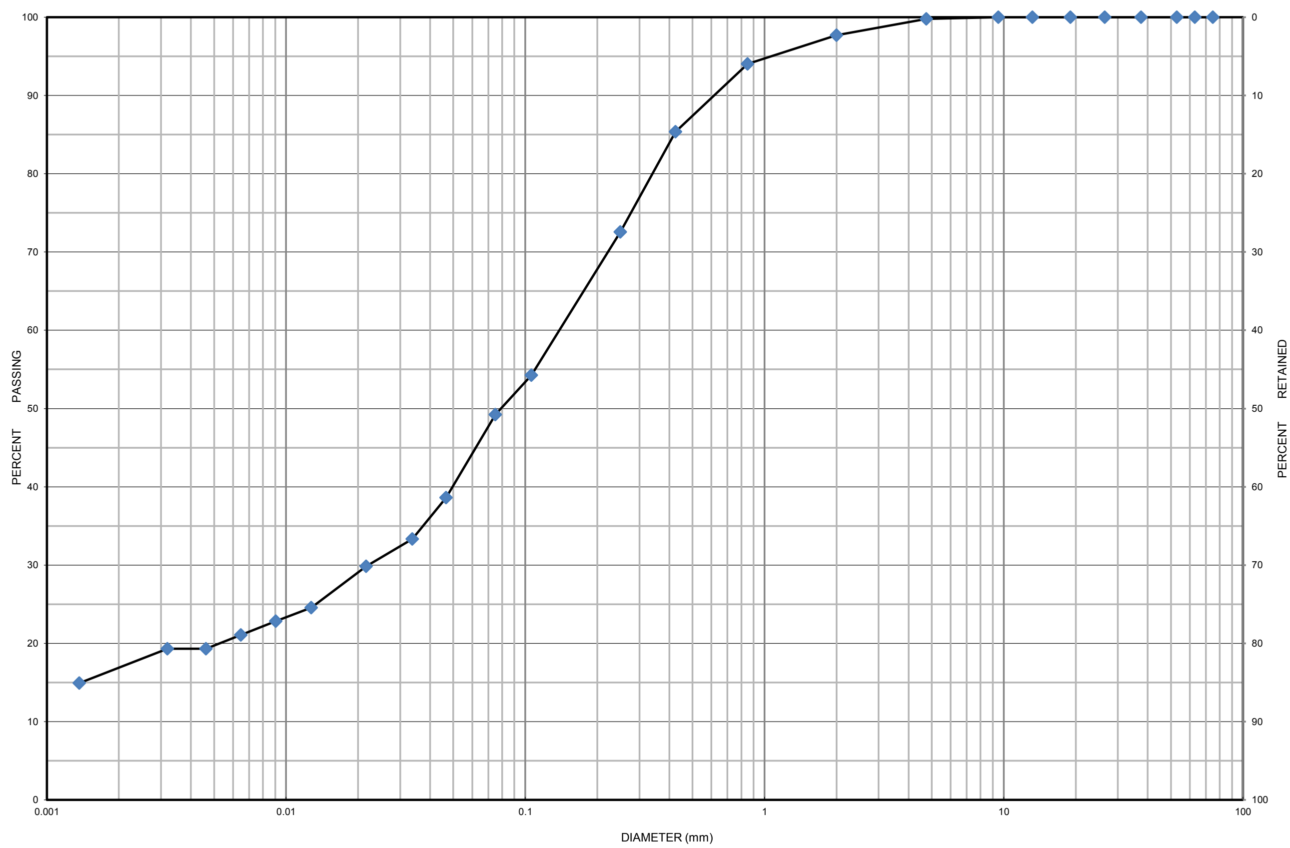
Date Issued: *November 24, 2022*

Grain Size Distribution Chart

Project Number:	*15101-001*	**Client:**	*Woodview Golf - Eric Challenger*		
Project Name:	*Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*				
Sample Date:	*November 4, 2022*	**Sampled By:**	*James Goodwin - Cambium Inc.*		
Location:	*TP 131-22 GS 2*	**Depth:**	*0.2 m to 0.4 m*	**Lab Sample No:**	*S-22-1684*

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM

CLAY	SILT	SAND			GRAVEL			BOULDERS
		FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 131-22	GS 2	0.2 m to 0.4 m	0	51	32	17	20.9

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silty Sand some Clay	SM	0.145	0.022	-	-	-

Additional information available upon request

Issued By: (Senior Project Manager)

Date Issued: *November 24, 2022*

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Grain Size Distribution Chart

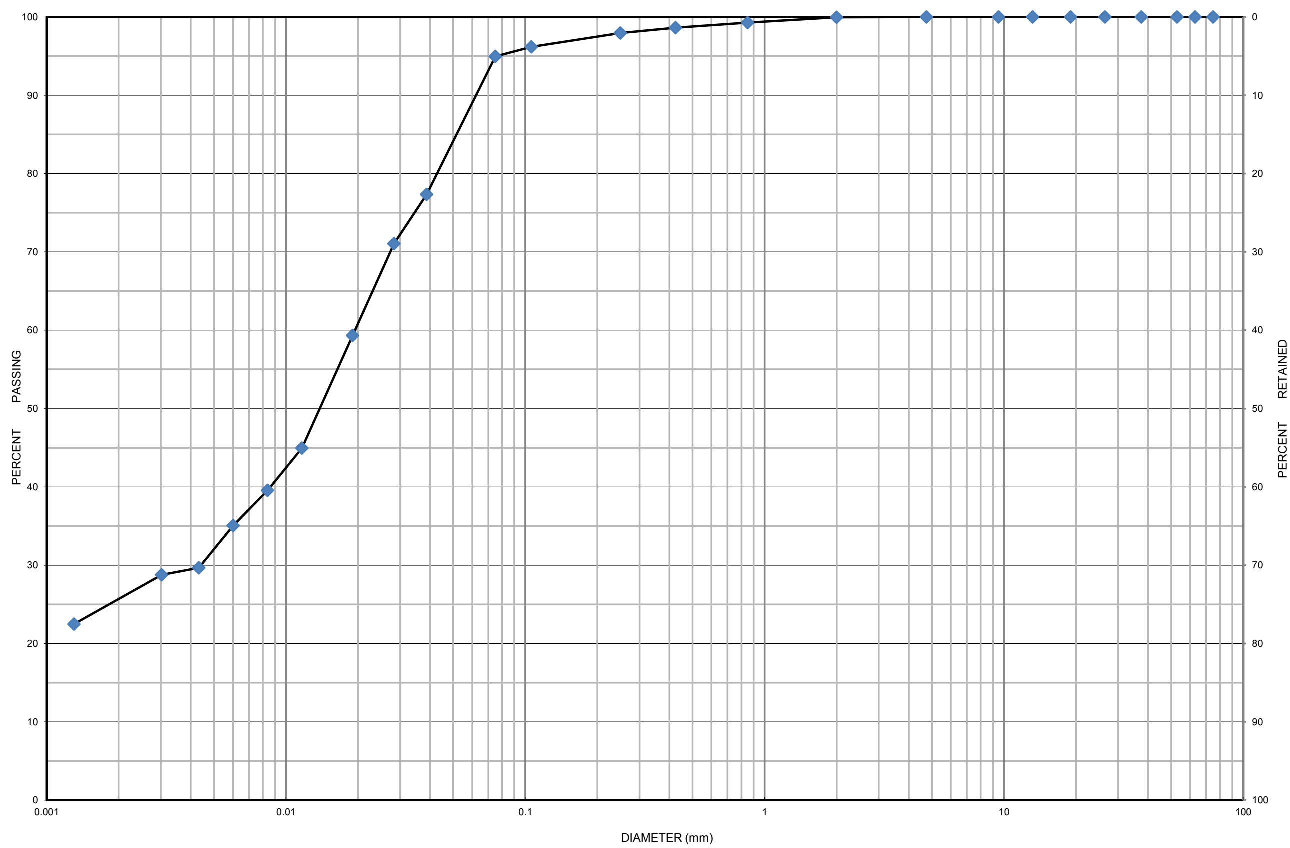
Project Number: *15101-001* **Client:** *Woodview Golf - Eric Challenger*

Project Name: *Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*

Sample Date: *November 4, 2022* **Sampled By:** *James Goodwin - Cambium Inc.*

Location: *TP 102-22 GS 2* **Depth:** *0.3 m to 0.4 m* **Lab Sample No:** *S-22-1677*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 102-22	GS 2	0.3 m to 0.4 m	0	5	69	26	23.6

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Clayey Silt trace Sand	ML	0.0190	0.0044	-	-	-

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Grain Size Distribution Chart

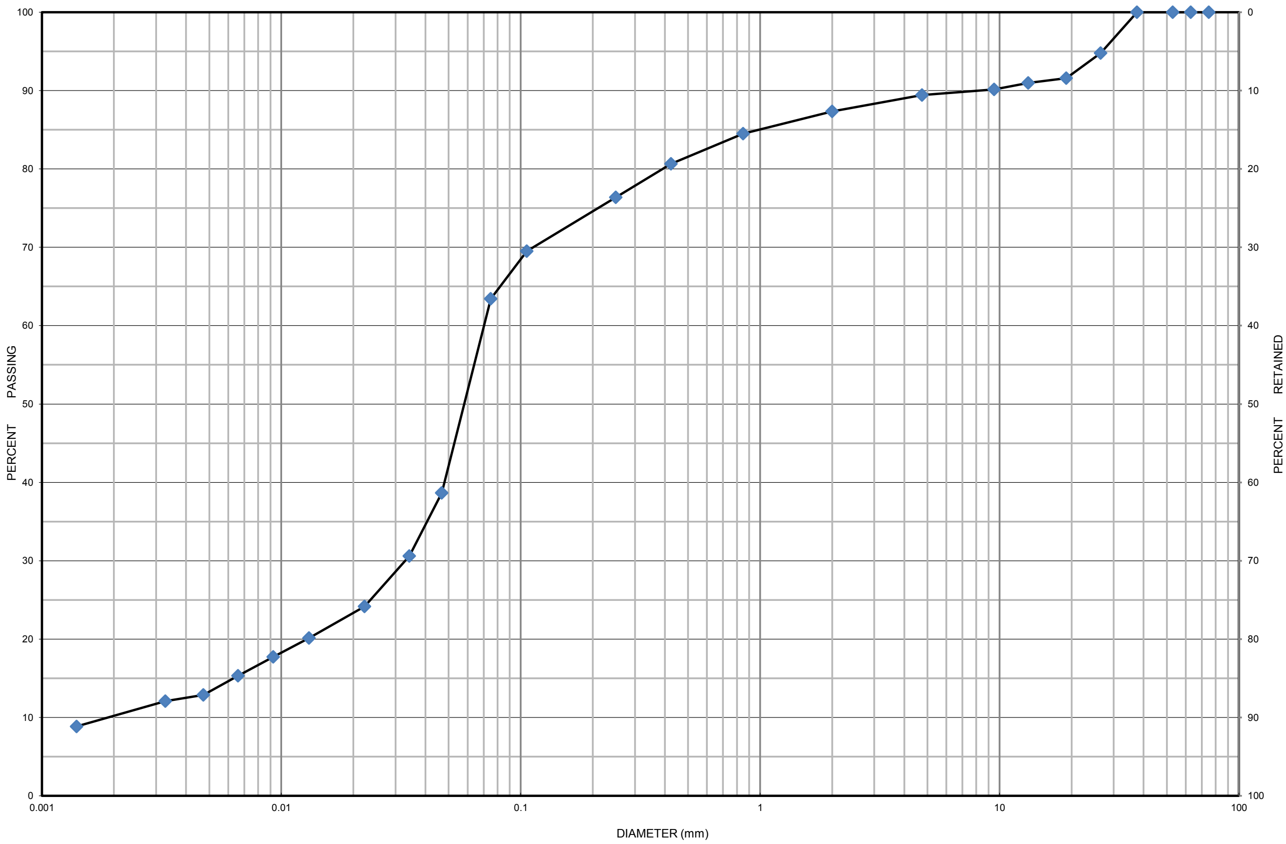
Project Number: *15101-001* **Client:** *Woodview Golf - Eric Challenger*

Project Name: *Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*

Sample Date: *November 4, 2022* **Sampled By:** *James Goodwin - Cambium Inc.*

Location: *TP 103-22 GS 2* **Depth:** *0.3 m to 0.5 m* **Lab Sample No:** *S-22-1678*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 103-22	GS 2	0.3 m to 0.5 m	11	26	52	11	8.8

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sandy Silt some Gravel some Clay	ML	0.0700	0.0330	0.0018	38.89	8.64

Additional information available upon request

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Grain Size Distribution Chart

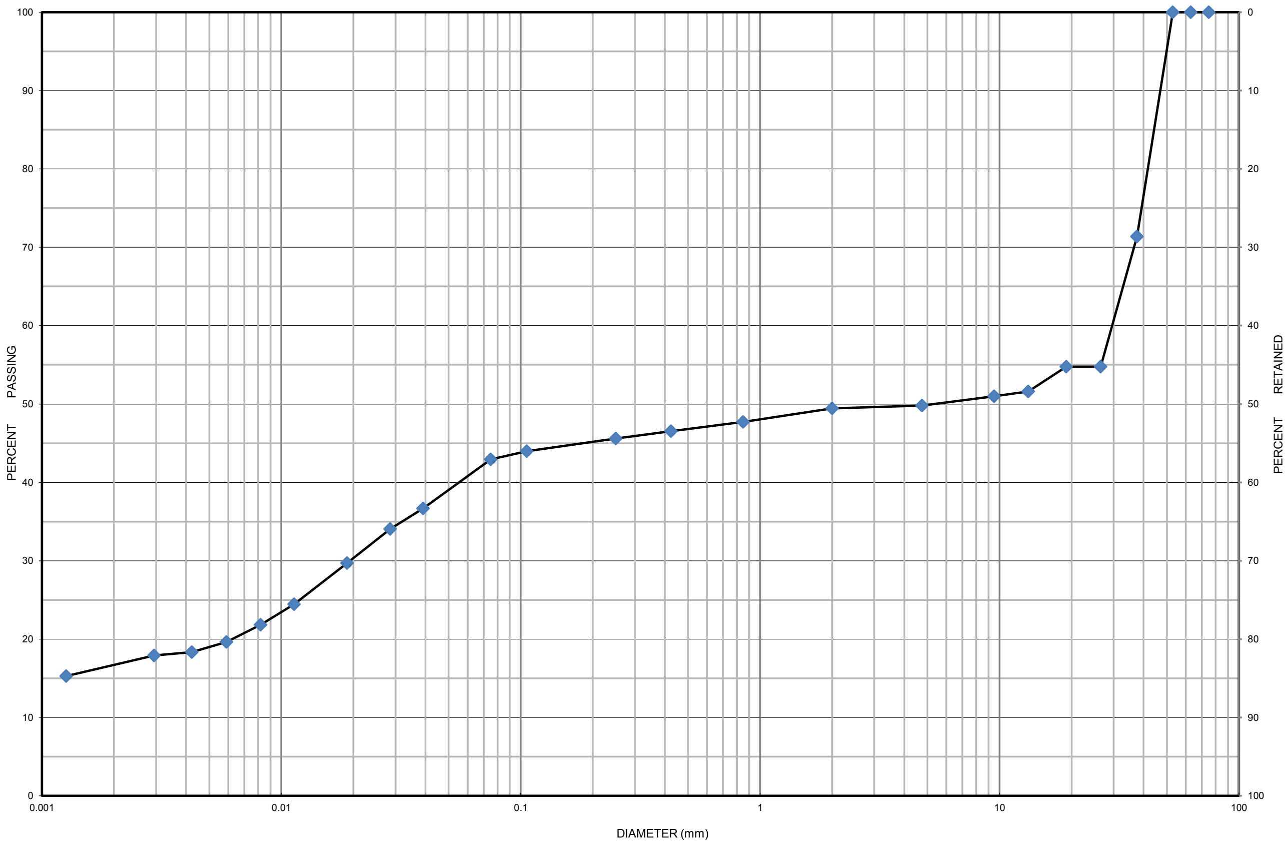
Project Number: *15101-001* **Client:** *Woodview Golf - Eric Challenger*

Project Name: *Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*

Sample Date: *November 4, 2022* **Sampled By:** *James Goodwin - Cambium Inc.*

Location: *TP 124-22 GS 2* **Depth:** *25.3 m to 1.1 m* **Lab Sample No:** *S-22-1680*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 124-22	GS 2	25.3 m to 1.1 m	50	7	26	17	10.0

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silty Gravel some Clay trace Sand	GM	30.000	0.019	-	-	-

Additional information available upon request

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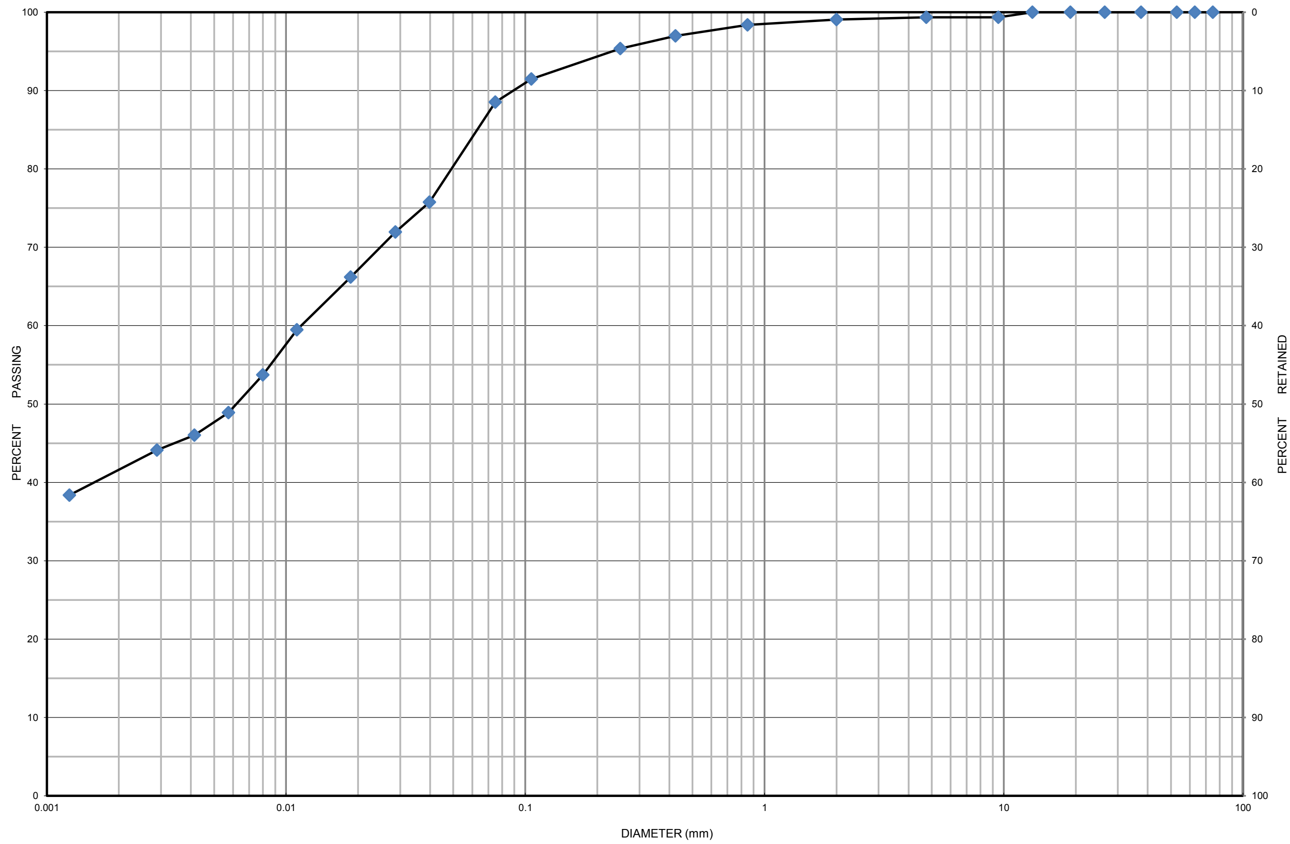
Grain Size Distribution Chart

Project Number:	*15101-001*	**Client:**	*Woodview Golf - Eric Challenger*		
Project Name:	*Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*				
Sample Date:	*November 4, 2022*	**Sampled By:**	*James Goodwin - Cambium Inc.*		
Location:	*TP 125-22 GS 2*	**Depth:**	*0.3 m to 0.7 m*	**Lab Sample No:**	*S-22-1681*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 125-22	GS 2	0.3 m to 0.7 m	1	11	46	42	34.4

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Silt and Clay some Sand trace Gravel	ML	0.012	-	-	-	-

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Grain Size Distribution Chart

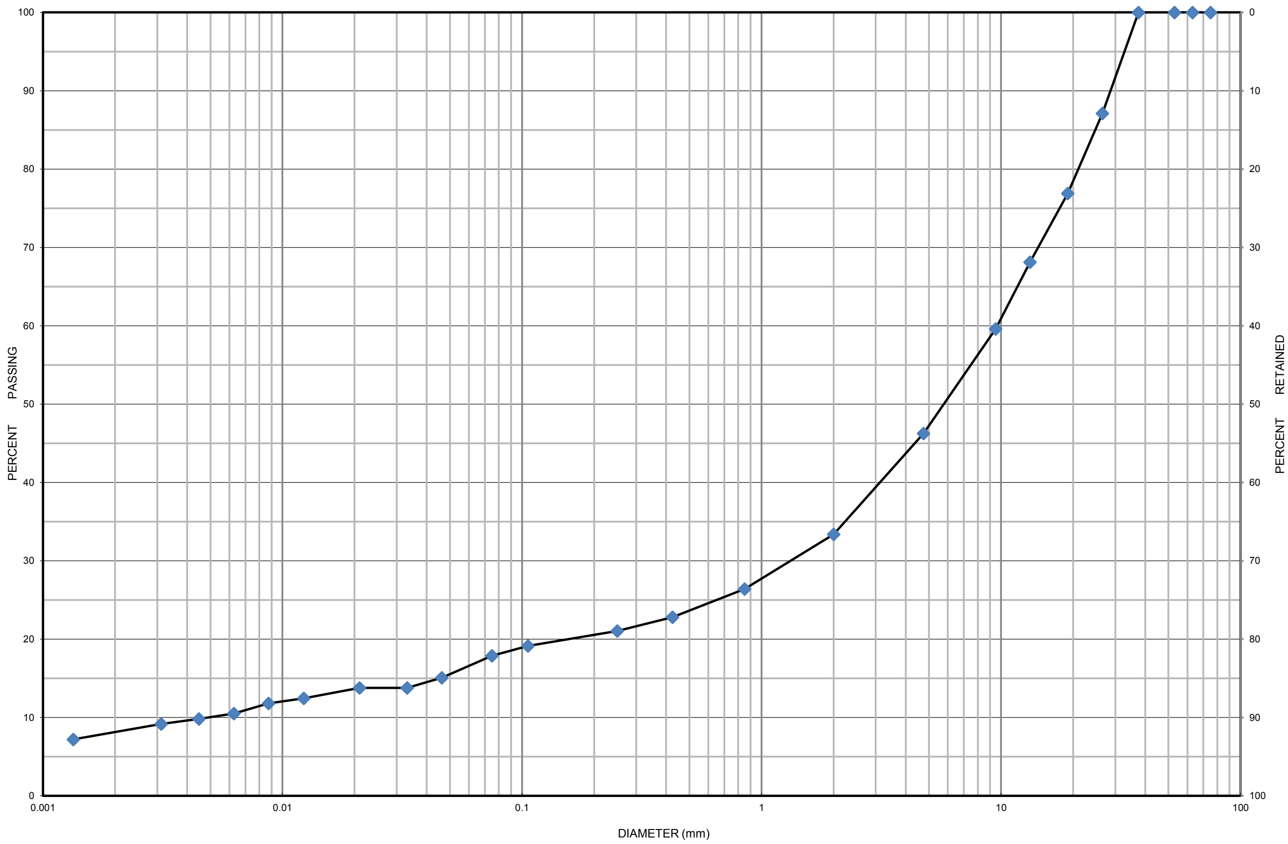
Project Number: *15101-001* **Client:** *Woodview Golf - Eric Challenger*

Project Name: *Hydrogeological & Geotechnical Assessment- Woodview Golf Subdivision*

Sample Date: *November 4, 2022* **Sampled By:** *James Goodwin - Cambium Inc.*

Location: *TP 126-22 GS 2* **Depth:** *0.3 m to 1.1 m* **Lab Sample No:** *S-22-1682*

UNIFIED SOIL CLASSIFICATION SYSTEM					
CLAY & SILT (<0.075 mm)	SAND (<4.75 mm to 0.075 mm)			GRAVEL (>4.75 mm)	
	FINE	MEDIUM	COARSE	FINE	COARSE



MIT SOIL CLASSIFICATION SYSTEM								
CLAY	SILT	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	BOULDERS
		SAND			GRAVEL			

Borehole No.	Sample No.	Depth	Gravel	Sand	Silt	Clay	Moisture
TP 126-22	GS 2	0.3 m to 1.1 m	54	28	10	8	9.5

Description	Classification	D_{60}	D_{30}	D_{10}	C_u	C_c
Sandy Gravel some Silt trace Clay	GM	9.6000	1.4000	0.0047	2042.55	43.44

Additional information available upon request

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