Stantec

GEOTECHNICAL INVESTIGATION

Preliminary Geotechnical Investigation for Proposed Industrial Development, 2509 Cedar Creek Road, Ayr, ON

January 16, 2024

Prepared for:
Cedar Creek Road Holdings Inc.

Prepared by:
Stantec Consulting Ltd.
100-300 Hagey Boulevard
Waterloo, ON N2L 0A4

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Prepared by:

Essa Nimer, EIT
Geotechnical Engineering Intern

Reviewed by:

Raid Khamis, P.Eng, MSc., PMP.,
Senior Associate, Geotechnical Engineering

Licensed Professional Engineer
2024/01/19
R. KHAMIS
100123563
Province of Ontario

Approved by:

Hassan Gilani, M.Sc. P. Eng
Senior Geotechnical Engineer

Licensed Professional Engineer
2024/01/16
S.H. GILANI
Province of Ontario

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1 INTRODUCTION

Stantec Consulting Ltd., (Stantec) has been retained by Cedar Creek Road Holdings Inc. (Client) to conduct a geotechnical investigation for the proposed land development located at 2509 Cedar Creek Road in Ayr, Ontario (Site). It is understood that the approximately 18-hectare Site is considered for purchase and redevelopment into seven (7) industrial lots, serviced by a stormwater management pond. The geotechnical investigation was initiated to provide preliminary geotechnical information in support of the planned development. The Site location is shown on **Drawing No. 1 – Site Location Plan**. An aerial view of the Site is shown on **Drawing No. 2 – Borehole Location Plan, Appendix B**

The majority of the Site is currently used for agricultural purposes, except for a residential dwelling with associated driveways and ancillary structures located in the northwest portion of the Site. Generally, site grades slope down in a southerly direction with an overall relief of approximately 13 m.

The purpose of the geotechnical investigation was to determine the subsurface soil and groundwater conditions at the Site, and to provide preliminary geotechnical design and construction recommendations for the proposed development.

Limitations associated with this report and its contents are provided in the statement included in **Appendix A**.

2 PROJECT AND SITE DESCRIPTION

The majority of the subject Site is currently vacant and used for agricultural purposes. The Site is bounded by Cedar Creek Road on its North, industrial developments on its East, and agricultural land on its west and south. A residential dwelling with associated driveways and ancillary structures is in the northwest portion of the Site. Generally, it was observed that the site grades slope down in a southerly direction with an overall grade change of about 13 m.

3 REGIONAL GEOLOGY

The site is located within the physiographic region of Southern Ontario known as the Waterloo Hills at its southern boundary with the physiographic region of the Horseshoe Moraines (Chapman and Putnam, 1984). The region is comprised of sandy hills, some of them being ridges of sandy till while others are kames or kame moraines, with outwash sands occupying the intervening hollows. In the main part of the region, numerous undrained depressions (kettles) infilled with swamp deposits exist between the hills. The approximately 750 km^2 physiographic region is comprised of a kame moraine, which is covered mostly with glaciofluvial and ice-contact sandy soils[1]. The surface sandy soils of variable thicknesses known as Waterloo Sands are underlain by alternate layers of stadial deposits i.e., glacial tills and sandy or clayey soils deposited during the intervening warmer climates.

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[1] Karrow, P.F. (1987): Quaternary Geology of the Cambridge Area, Southern Ontario; Ontario Geological Survey Preliminary Map P.2508, Scale 1:50,000.

The Site is underlain by the Upper Silurian Salina Formation[2], which is comprised of grey-brown Dolomite, interbedded with grey and red shale, minor limestone**,** and evaporites gypsum and salt; salt is present at deeper depths. The bedrock formation is expected to be at elevations of approximately 266 m to 274 m Above Mean Sea Level (AMSL)[3]**,** approximately 40 m to 50 m below the existing ground surface in the general Site area**.**

4 SCOPE OF WORK

The scope of work (SOW) carried out for the geotechnical investigation comprised of the following tasks:

- Contact the public utility authorities to confirm the locations of major public utilities.
- Retain a private utility locate firm to scan the intended borehole locations and mark any buried services or utilities within 3 m of these locations.
- Advance a total of eight (8) geotechnical boreholes to the following depths:
  - Five (5) boreholes advance to 6.71 m below ground surface.
  - Two (2) boreholes advance to 8.53 m below ground surface and equipped with monitoring wells.
  - One (1) borehole advanced to 14.33 m below ground surface
- Equip all boreholes with monitoring well to the specified termination depth.
- Collect soil samples in each borehole at regular intervals by driving a split tube sampler in accordance with the methods and procedures described in ASTM D1586. The samples obtained will be placed in moisture-proof containers and transported to our geotechnical materials testing laboratory for classification and testing.
- Record the presence and depth (where encountered) of free groundwater in the open boreholes.
- Record the ground surface locations at the borehole locations and coordinates of the boreholes using survey equipment.

The Boreholes\Monitoring Well Location plan is shown on Drawing No. 2, Appendix B.

4.1 UTILITY LOCATES

Prior to commencing the field investigation, Stantec contacted Ontario One Call to locate the underground public utilities to provide utility clearances. In addition, Stantec retained the services of a utility locate company, Premier Locates, to provide private utility locate services to identify any traceable underground utilities not identified by the public locates. A total of eight locations were marked on site and cleared from any underground utilities.

4.2 FIELDWORK

The borehole drilling program was conducted from February 15 to 18, 2022, including the advancement of eight (8) boreholes (BH/MW01-22 through BH/MW08-22) at the locations shown on the appended **Drawing 2 – Borehole location plan**. The boreholes were advanced to depths of 6.71 m below ground surface (BGS) to 14.3 m BGS.

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[2] D.F. Hewitt (1972), Paleozoic Geology of Southern Ontario, Geological Report 105, Map 2254, scale 1:1,013,760 p10

[3] Miller, R.F., Farrel, Lorraine, and Karrow, P.F., (1979), Bedrock Topography of the Cambridge Area, Southern Ontario Geological Survey Prelim. Map P. 1985, Bedrock Topography Ser., Scale 1:50,000 Geology 1978

Soil samples were recovered at regular intervals using a 50 mm (outside diameter) split-tube sampler by conducting Standard Penetration Tests (SPTs) in accordance with the procedures outlined in ASTM specification D1586. Soil samples recovered from the boreholes were placed in moisture proof bags. All samples were returned to Stantec's laboratory for geotechnical classification. Groundwater conditions were recorded on 5 occasions since the conclusion of fieldwork, between March 10, 2022, to November 22, 2023.

The monitoring wells installed in all boreholes were constructed using a 50 mm inside diameter, Schedule 40 PVC pipe with a 3-meter No. 10 slot screen (0.01-inch slot) with a tower casing above ground surface. The annular space between the monitoring well pipe and surrounding soils was backfilled with silica sand to a maximum of 0.3 m above the top of the screen and the remainder of the annular space was filled with bentonite. The well cluster tag number for the monitoring wells is # A311058, tagged on borehole BH/MW02-22. The installation details are shown on the borehole logs included in **Appendix C – Borehole Records.**

The borehole survey information was collected by Stantec field personnel, using a Trimble R12 GPS on March 10th, 2022. Ground surface elevations at the borehole locations referenced to a geodetic datum and approximate UTM coordinates (Zone 17 NAD 83) and are shown in Table 4.1 below:

Table 4.1: Borehole Location and Elevation Summary

Borehole No.	Borehole depth	UTM Coordinates (Zone 17 NAD83)		Ground Surface Elevation
	(m)	Easting (m)	Northing (m)	(m)
BH/MW 01-22	8.53	4798023	546034	309.67
BH/MW 02-22	14.33	4798068	546169	314.89
BH/MW 03-22	8.23	4797909	546107	308.14
BH/MW 04-22	6.40	4797789	546101	311.83
BH/MW 05-22	6.71	4797572	546107	306.74
BH/MW 06-22	6.71	4797690	546294	306.20
BH/MW 07-22	6.71	4797375	546154	306.38
BH/MW 08-22	5.18	4797336	546399	301.79

It was observed that the ground sloping went from North to South, with an elevation change of approximately 13 m above sea level (ASL) across the site.

4.3 LABORATORY TESTING

All soil samples obtained from the boreholes were subjected to visual and tactile examination on return to the geotechnical and construction materials testing laboratory. The geotechnical laboratory testing was completed on a number of samples, as shown in Table 4.2:

Table 4.2: Geotechnical Laboratory Testing Program

Laboratory Test	Number of Samples Tested
ASTM D2216-10 – Natural Moisture Content	14
ASTM D422-63 (2007) – Grain Size Distribution with Hydrometer	4
ASTM D4318-10 – Atterberg Limits	1

The results of the laboratory tests are discussed in the text of this report and are plotted on the respective borehole logs included in **Appendix C.** Figures illustrating the results of the grain size distribution tests and Atterberg Limits tests are included in **Appendix D.**

Unless specific instructions are received to the contrary, the samples will be discarded three (3) months after issue of this report.

5 RESULTS OF INVESTIGATION

5.1 FRAME OF REFERENCE

The soils encountered in the boreholes and reported herein have been classified in accordance with the Unified Soil Classification System as defined in ASTM D2487 per Unified Soil Classification System (USCS) and D2488 per visual-manual method.

It should be noted that the internal diameter (I.D.) of the SPT sampler is 38 mm and hence the grain size test results and soil classifications may not reflect the entire gravel size fraction which extends to 75 mm diameter. The presence of cobbles (particles from 75 mm to 300 mm) and boulders (particles > 300 mm) were inferred to be present in specific stratum and are described separately from the gravel content.

It should also be noted that the stratigraphic boundaries shown on the borehole logs are inferred from non-continuous sampling and should be considered approximate only.

5.2 OVERVIEW OF CONDITIONS

The subsurface conditions encountered at the borehole locations are provided on the Borehole Records in **Appendix C**, along with an explanation of the symbols and terms used in the Borehole Records.

The following paragraphs provide additional information on the soil strata encountered in the boreholes. The following is intended to summarize the conditions encountered; the Borehole Records provided in **Appendix C** should be used as the primary source of information supplemented by the information provided in the following sections. The soil conditions shown on the records are a direct extraction from the associated boreholes.

5.3 TOPSOIL

Approximately 150 mm to 200 mm thick topsoil was present at the ground surface at all borehole locations. The topsoil comprised of silty sand to sandy silt containing roots and organics. The thickness of the topsoil can be attributed to the tilling of the surface for agricultural purposes. SPT N-values in the sand varied from 7 to 22 blows indicating a loose to compact relative density. The topsoil was described as frozen to moist during fieldwork.

5.4 SAND (SP, SP-SM, SM)

Sand (SP, SP-SM, SM) with variable silt and gravel content was encountered in the majority of the boreholes (BH/MW01-22, BH/MW03-22, BH/MW05-22, BH/MW06-22, and BH/MW08-22) underlying the topsoil or interlayered with the gravel deposits. The sand deposit was noted to contain occasional to numerous cobbles. SPT N-values in the sand varied from 3 to greater than 50 blows indicating very loose to very dense relative densities. In general, the compactness of the sand increased with depth. The sand was described as damp to saturated during fieldwork. Laboratory determined moisture contents ranged from 3% to 21%.

Grain size distribution analyses were conducted on one (1) sample. The results of the tests are shown in Table 5.1 below.

Table 5.1: Summary of Grain Size Distribution Results

Borehole	Depth	Grain Size (%)			Wn (%)	Soil Classification
	(m BGS)	Gravel	Sand	Fines		
BH/MW 01-22	6.4	15	78	7	14	Poorly Graded Sand (SP-SM) with Silt and Gravel

Based on the results of the test referenced above and visual inspection of the samples, the samples obtained are classified as poorly graded sand with gravel (SP), in accordance with the USCS (ASTM D2487).

5.5 GRAVEL WITH SILT AND SAND (GW-GC)

Sand and Gravel (SP-GP) and gravel (GW-GC) with silt and sand were contacted at BH/MW02-22, BH/MW04-22 and BH/MW07-22 underlying the topsoil layer or silt. Occasional to frequent cobbles and/or boulders were noted within the sand and gravel and gravel deposits. SPT-N values ranged from 6 to greater than 50 blows indicating variable loose to very dense relative densities. It is noted that at borehole BH/MW02-22 loose conditions extended to 6.1 m BGS.

Grain size distribution analyses were conducted on one (1) sample. The results of the tests are shown in Table 5.2 below.

Table 5.2: Summary of Grain Size Distribution

Borehole	Depth	Grain Size (%)			Wn (%)	Soil Classification
	(m BGS)	Gravel	Sand	Fines		
BH/MW 04-22	4.9	51	40	9	3	Well Graded Gravel (GW-GC) with Silt and Sand

Based on the results of the test referenced above and visual inspection of the samples, the samples obtained classified as Well graded gravel with silt and sand(GW-GC), in accordance with the USCS (ASTM D2487).

5.6 SILT (SM)

Layers and/or deposits of sandy Silt (ML) and silt (ML) with trace to some sand and clay were contacted at boreholes BH/MW01-22, BH/MW02-22, BH/MW03-22, and BH/MW06-22. SPT-N values of 5 to 42 blows per 300 mm indicated loose to dense relative densities. In general, the sandy silt and silt was compact to dense with loose conditions observed within the upper 2.3 m at borehole BH/MW02-22. At the time of fieldwork, the sandy silt and silt were described as moist to saturated. The saturated silt was dilatant. Laboratory determined moisture contents ranged from 16% to 17.5%.

Grain size distribution analyses were conducted on one (1) sample. The results of the tests are shown in Table 5.3 below.

Table 5.3: Summary of Grain Size Distribution and Atterberg Limits Analyses Results

Borehole	Depth	Grain Size (%)			Wn (%)	Soil Classification
	(m BGS)	Gravel	Sand	Fines		
BH/MW 03-22	4.9	0	7	93	16	Silt (ML)

Based on the results of the test referenced above and visual inspection of the samples, the samples obtained are classified as Silt (ML), in accordance with the USCS (ASTM D2487).

5.7 SILTY CLAY (CL-ML)

A cohesive silty clay (CL-ML) deposit was contacted at borehole BH/MW03-22 underlying the silt at 6.2 m BGS and extended below the borehole termination depth of 8.2 m BGS. The silty clay contained saturated sand layers. At the time of sampling the silty clay was about the plastic limit to wetter than plastic limit (APL to WTPL). A laboratory determined moisture content of 21% was reported. SPT N-values of 18 and 23 blows per 300 mm and approximate undrained shear strength of approximately 175 kPa to 100 kPa as determined by handheld pocket penetrometer, indicated very stiff conditions.

Grain size distribution analyses and Atterberg limits were conducted on one (1) sample. The results of the tests are shown in Table 5.4 below.

Table 5.4: Summary of Grain Size Distribution and Atterberg Limits Analyses Results

Borehole	Depth	Grain Size (%)			Wn (%)	Atterberg Limits (%)			Soil Classification
	(m BGS)	Gravel	Sand	Fines		PL	PI	LI	
BH/MW 03-22	7.9	0	6	94	21	15	4	1.5	Silty Clay (CL-ML)

Notes:
1. Fines denote fraction passing the No. 200 sieve.
2. Wn denotes the natural water content.
3. LL, PL, and PI denote Liquid Limit, Plastic Limit and Plasticity Index, respectively.
4. LI denotes Liquidity Index = (Wn - PL)/PI
5. Soil classification in accordance with USCS (ASTM D2487)

A review of the results shows that the Liquidity Index of the soils is generally less than 0.25, indicating that the samples represent a deposit, which is in a very stiff state of consistency. This observation matches well with the recorded SPT ‘N’ values.

Based on the results of the test referenced above and visual inspection of the samples, the samples obtained from are classified as Silty Clay (CL-ML), in accordance with the USCS (ASTM D2487).

5.8 GROUNDWATER

A total of eight (8) monitoring wells were installed as a part of the geotechnical investigation. Wet to saturated deposits were noted at the time of drilling at all boreholes approximately 5 m below ground surface. Water level was measured on five (5) different occasions between March 10, 2022, to November 22, 2023. It was observed that the groundwater was encountered within the boreholes drilled at the eastern portion (BH/MW 01-22, BH/MW 02-22 and BH/MW 03-22) at elevations ranging from 303.6 m to 305.8 m ASL. However, water was not encountered when measured for the remainder of the boreholes across the site.

Perched groundwater conditions following heavy rain or snow melt in the Spring should be expected within the upper loose deposits or above the silty soils at shallower depths. Seasonal fluctuations and variations should be expected. A hydrogeological report will be issued under a separate cover to discuss the findings from the investigation. The measured water levels are provided in Table 5.5.

Table 5.5 Measured Groundwater Levels

Borehole	Ground surface Elevation	Well Depth (m BGS)	Water Level		
	(m AMSL)		Date	Depth (m BGS)	Elevation (m AMSL)
BH/MW 01-22	309.7	7.95	10-Mar-22	5.14	304.5
			2-May-22	4.88	304.8
			3-Nov-22	5.34	304.3
			27-Jan-23	5.50	304.2
			22-Nov-23	5.06	304.6
BH/MW 02-22	314.9	13.48	11-Mar-22	10.23	304.7
			2-May-22	10.28	304.6
			3-Nov-22	11.32	303.6
			27-Jan-23	-	-
			22-Nov-23	10.83	304.1
BH/MW 03-22	308.1	7.08	11-Mar-22	2.32	305.8
			2-May-22	2.81	305.3
			3-Nov-22	3.56	304.6
			27-Jan-23	3.63	304.5
			22-Nov-23	3.13	305.0
BH/MW 04-22	311.8	5.94	11-Mar-22	Dry	-
			2-May-22	Dry	-
			3-Nov-22	Dry	-
			27-Jan-23	Dry	-
			22-Nov-23	Dry	-
BH/MW 05-22	306.7	5.91	11-Mar-22	Dry	-
			2-May-22	Dry	-
			3-Nov-22	Dry	-
			27-Jan-23	Dry	-
			22-Nov-23	Dry	-
BH/MW 06-22	306.2	5.89	11-Mar-22	Dry	-
			2-May-22	Dry	-
			3-Nov-22	Dry	-
			27-Jan-23	Dry	-
			22-Nov-23	Dry	-
BH/MW 07-22	306.4	6.03	11-Mar-22	Dry	-
			2-May-22	Dry	-
			3-Nov-22	Dry	-
			27-Jan-23	Dry	-
			22-Nov-23	Dry	-
BH/MW 08-22	301.8	5.02	11-Mar-22	Dry	-
			2-May-22	Dry	-
			3-Nov-22	Dry	-
			27-Jan-23	Dry	-
			22-Nov-23	Dry	-

6 GEOTECHNICAL ENGINEERING DESIGN AND RECOMMENDATION

The Site located at 2509 Cedar Creek Road in Ayr, Ontario is considered for redevelopment into seven (7) industrial lots. An access roadway from Cedar Creek Road to each lot is planned along the western property limits.

It's our understanding that the proposed industrial developments will comprise one to two strorey slab-on-grade(no basement) structures, serviced by internal roads, parking lots and buried utilities.

The geotechnical comments, discussion, and recommendations are provided in the following sections for the preliminary design and construction of the planned residential/commercial/institutional development. Based on the results of the geotechnical investigation, the subject site is considered suitable for the proposed development from a geotechnical point of view, subject to the recommendations discussed in the following sections.

6.1 GRADING

It is anticipated that site grading activities will be required to prepare the lands for the proposed development. Prior to grading activities, the existing residential structure should be demolished, and all services, and concrete (including foundations and floor slabs) should be completely removed, and resulting voids filled with engineered fill, if located within the future settlement sensitive areas. The existing topsoil layer and any organics or existing fill (i.e., within developed portion of the lands) should be removed from both cut and fill areas prior to any grading activities. The existing surficial topsoil may be used in landscaped areas or removed from site.

Following stripping, the subgrade in areas of proposed fill is to be inspected by geotechnical personnel to ensure that all unsuitable materials are removed. All very loose/soft soils or wet zones identified during site preparation or during general construction activities, are to be removed and replaced with approved engineered fill below proposed buildings or subgrade fill below proposed pavement structures. Loose soils were noted at borehole BH/MW02-22 to 6.1 m BGS. This area must be thoroughly assessed by geotechnical personnel to determine if any sub-excavation or soil improvement is needed prior to fill placement.

The exposed subgrade surface should be proof rolled and compacted across the entire area of the planned development. The proof rolling program should be undertaken using large, non-vibratory compaction equipment having a minimum static weight of 10 tonnes. This will provide a uniform, compact surface that will minimize the potential for infiltration of precipitation and ground surface runoff and promote overland drainage at the ground surface. The proof rolling program should consist of a minimum of five passes per unit area to provide a uniform surface for construction. Vibratory compaction is not recommended for silty soils, due to the potential for subgrade pumping that could occur if perched groundwater is present near the surface.

Engineered fill below buildings must extend horizontally 1 m beyond the edge of proposed footings, and then downwards and outwards at a slope of 1 horizontal to 1 vertical to competent soil. Engineered fill will need to be benched into any native slope steeper than 3 horizontals to 1 vertical. The benching should be excavated with heights matching the engineered fill lift thickness.

The on-site sand and/or gravel soils excavated from above the groundwater table are generally considered suitable for reuse as engineered fill and subgrade fill. Silt soils should be excluded from reuse as engineered fill; however, may be suitable for reuse as subgrade fill following inspection by geotechnical personnel. Soils considered for reuse must have a moisture content within 2% of the optimum moisture content of the soil. The native soils typically have a variable low to high moisture content. Depending on the moisture contents at the time of construction, blending and drying of these native soils may be required prior to reuse on-site. If work is carried out in dry weather, then water may have to be added to aid in compaction of sandy and gravelly soils. Cobbles or boulders should be separated from the material prior to its reuse as fill.

If additional imported fill materials are required for raising the grade on site, it is recommended that granular materials or materials with characteristics similar to the native soils on site be imported for this purpose. Imported Granular 'B' (recommended) or OPSS SSM are recommended for use as engineered fill below buildings. Other soil types may also be suitable but must be tested and confirmed as acceptable by a geotechnical engineer prior to being imported to site.

It is expected that groundwater level is situated at elevations between 303.6 m to 305.4 m ASL in the northern portion of the site. Where wet soils and/or soils with low internal strength are exposed at the subgrade level, placement of a woven geotextile followed by placement of imported granular soils such as OPSS.MUNI 1010 Granular B could be considered to create a stable working base for additional fill placement using on-site soils.

Engineered fill below buildings should be placed in maximum 300 mm thick lifts and compacted to at least 98% standard Proctor maximum dry density (SPMDD). Subgrade fill below roadways or parking areas must be placed in maximum 300 mm thick lifts and compacted to at least 98% SPMDD.

It is advised that engineered fill operations are supervised and tested by a qualified geotechnical technician during the fill process after compaction of each lift to ensure requirements are met.

6.2 DEPTH OF FROST PENETRATION

The design depth of frost penetration in the general Site area is 1.2 m in accordance with the Ontario Provincial Standard Drawing (OPSD) 3090.101. Therefore, a permanent soil cover of at least 1.2 m or its thermal equivalent synthetic insulation is required for frost protection of foundations in unheated areas.

During winter construction, exposed surfaces to support foundations must be protected against freezing by means of loose straw and tarpaulins, heating, etc.

6.3 CONVENTIONAL SHALLOW FOUNDATIONS

It is anticipated that the industrial lots will be developed including industrial use buildings. Details regarding building size and location as well as finished floor elevations are not available.

Loose soils were noted at borehole BH/MW02-22 to a depth of 6.1 m below current grades. Additional investigation efforts are required to further support the detailed design stage as well as the site review plan for individual lots. The additional investigative measures are recommended in this area to investigate the extent of the loose deposit. Moreover, depending on proposed site grades as well as site use, soil improvement may be required. The following foundation design recommendations are considered preliminary.

The preliminary geotechnical resistances at ULS and reactions at SLS for the case of conventional spread and strip footing foundations placed on engineered fill constructed on approved compact native soils or foundations placed on marginally compact native soils are provided in Table 3 below. Higher bearing resistances would be available within the compact to dense native soils generally contacted below 1.5 m to 7.6 m BGS as provided in Table 6.2 below. A minimum soil cover of 1.2 m required for protection against frost was considered in the evaluation of the preliminary ULS values.

Table 6.1: Preliminary Conventional Spread and Strip Footings on Engineered Fill or Marginally Compact Native Soils

Footing Width (m)	Factored ULS Resistance (kPa)	SLS Reaction (kPa)
Spread Footings: 1.0 to 3.0	225	150
Strip Footings: 0.45 to 0.9	180	150

Table 6.2: Preliminary Conventional Spread and Strip Footings on Compact to Dense Native Soils

Footing Width (m)	Factored ULS Resistance (kPa)	SLS Reaction (kPa)
Spread Footings: 1.0 to 3.0	350	250
Strip Footings: 0.45 to 0.9	240	240

The geotechnical bearing resistance, ULS incorporates a resistance factor of 0.5. The geotechnical reaction, SLS, is the bearing pressure that corresponds to 25 mm of total settlement. Additional bearing pressures can be provided for larger footing widths, if required. Higher bearing pressures may be available, and this can be confirmed in the detailed investigation phase, including site specific investigations.

It is recommended that a 75 mm mud slab is placed on the prepared subgrade following inspection to minimize the potential for founding soil disturbance during rebar placement and forming.

Foundation walls should be backfilled with free-draining granular material such as OPSS Granular ‘B’ Type I, or a manufactured drainage layer should be provided. The exterior (perimeter) wall backfill should be placed in loose lifts having a maximum thickness of 300 mm. Each lift should be uniformly compacted using suitable compaction equipment for the purpose intended, to achieve a minimum of 95% of the material’s SPMDD.

Foundations for buildings, abutments, piers, retaining walls, sign supports, lighting poles, etc., should be protected from frost action by a minimum soil cover of 1.2 m, or be provided with equivalent protection using manufactured insulation. Where construction is undertaken during winter conditions, the footing subgrade must be protected from freezing.

6.4 FLOOR SLABS

6.4.1 Slab-on-Grade

The preliminary grading plan shows that each lot will include seven (7) industrial facilities as well as a SWM pond. Loose soils were noted at borehole BH/MW02-22 to a depth of 6.1 m below current grades. Additional boreholes are recommended in this area to investigate the extent of the loose deposit. Depending on proposed site grades as well as site use, soil improvement may be required for that location. The subgrade is expected to consist of approved engineered fill or compact to dense native soils consisting of mainly sandy soils.

A layer of free-draining granular material such as OPSS.MUNI 1010 Granular ‘A’, a minimum of 200 mm thick, should be placed immediately beneath the slab. This layer will act as a leveling course and as a moisture break under the floor slab(s). This material should be compacted to achieve a minimum of 100% of the material’s SPMDD.

Floor slabs founded on engineered fill or undisturbed inorganic compact to dense native soils should be designed using a preliminary modulus of sub-grade reaction of 30 MPa/m.

The modulus of sub-grade reaction provided is based on a loaded area of 0.3 m by 0.3 m. The modulus should be adjusted for the actual size of the loaded area. In order to achieve the modulus of sub-grade reaction above it has been assumed that the groundwater will be a minimum of 1.0 m below the base of the leveling course.

All slabs-on-grade should be independent from the load-bearing walls and columns.

6.5 SEISMIC SITE CLASS

The seismic Site Class value, as defined in Section 4.1.8.4 of the 2012 OBC, as amended, contains a seismic analysis and design methodology which uses a seismic site response and site classification system defined by the average shear stiffness of the upper 30 meters of the ground below the foundation level. The recommended site classification for seismic site response for this Study Area for shallow foundations founded on native compact soils is Site Class D.

6.6 EXCAVATIONS AND BACKFILL

6.6.1 Temporary Excavations – Soil Overburden

Temporary excavations for the proposed development must be carried out in accordance with the latest edition of the Occupational Health and Safety Act (OHSA). The native soil classification for excavation as per the table 6.3:

Table 6.3: Soil Types as per OHSA

Soil Type	Above groundwater	Below groundwater
Native soil (predominantly sandy soils)	3	4

Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the regulation requirements. The regulation stipulates safe excavation slopes by soil type as per table 6.4.

Table 6.4: Excavation Slopes for Each Soil Type as per OHSA

Soil Type	Base of Slope	Slope inclination
1	Within 1.2 meters of bottom of excavation	1H:1V
2	Within 1.2 meters of bottom of excavation	1H:1V
3	From Bottom of excavation	1H:1V
4	From Bottom of excavation	3H:1V

The soils above the groundwater table can be considered Type 3 for temporary excavation purposes and can be excavated at 1H:1V. Soils affected by groundwater seepage must be classified as Type 4 soil. The maximum excavation side slope for a Type 4 soil is 3H:1V (Horizontal: Vertical) in accordance with the OHSA regulation.

Stockpiling of any materials adjacent to excavations should be avoided. Similarly, traffic should not be permitted in proximity to open excavations. For this purpose, it is recommended that all storage of materials and traffic be restricted from a 3 m wide strip around the excavations, measured from the crest of the excavation designed and constructed in accordance with the OH&S Act.

If space is restricted such that the side slope cannot be safely cut back in accordance with the OH&S Act & Regulations, if sloughing and cave-in are encountered in the excavations, or if the excavations are to remain open for a longer period, an engineered shoring system should be used for the approximately up to 7 m deep bulk excavation for the proposed below grade levels. To avoid tiebacks, a soldier pile-lagging system would be suitable for up to 7 m deep excavation.

6.6.2 Groundwater Control

Based on the information revealed during the investigation, it is considered that conventional filtered sump pumping should be applicable to control localized seepage that may occur for an excavation at depths of 1.0 to 5.0 m below the existing grade across the site for clayey soils. If significant groundwater ingress is encountered is silty and sandy soils, then positive dewatering methods such as vacuum well points may be required. Services of a specialist dewatering contractor may be required.

Excavations extending more than 0.5 m below the stabilized groundwater in the predominant sand deposits or excavations into saturated gravel deposits will likely require a positive dewatering system.

6.7 SITE SERVICING

The predominant subgrade soils beneath the service pipes will consist of compact sandy soils, which would provide suitable supports to the proposed service utility pipes. The native soils should be removed and replaced with granular fill (OPSS Granular A or OPSS Granular B Type II) compacted to 100% SMPDD. Prior to installation of the services, the subgrade should be inspected by an experienced geotechnical engineer/technician. If any very loose or soft areas are detected during inspection, they should be excavated and replaced with compacted granular material such as OPSS.MUNI 1010 Granular A or Granular B Type II.

The pipe bedding for the services should be conventional Class B pipe bedding comprising a minimum 150 mm thick layer of OPSS.MUNI 1010 Granular ‘A’ aggregate below the pipe invert. The bedding course may be thickened if portions of the subgrade become wet during excavation. OPSS.MUNI 1010 Granular A type aggregate should be provided around the pipe to at least 300 mm above the top, and the bedding should be compacted to 98% SPMDD. Service lines installed outside of heated areas should be provided with a minimum 1.2 m of soil cover or equivalent insulation for frost protection.

Additional specific comment to the design of buried services and utilities in view of the subsurface conditions encountered in the boreholes and in consideration of good industry practice is provided as follows.

6.8 TRENCH BACKFILL

Bedding for services should consist of OPSS Granular ‘A’ material. In general, a minimum of 150 mm of bedding and 300 mm of cover material is recommended. The bedding and cover material should be compacted to achieve a minimum of 98% of the material’s SPMDD. The bedding and cover on each side of the pipe should be completed simultaneously and at no time should the difference from one side of the pipe to the other exceed 200 mm.

These recommendations should be confirmed with the pipe manufacturer and care must be taken to avoid incurring damage to the services. Pipe manufactures may have additional/alternative requirements that should be reviewed by the Designer and Contractor prior to installation of the services.

The trenches above the specified pipe bedding should be backfilled with inorganic soils that are not excessively wet placed in 200 mm thick lifts and compacted to at least 98% SPMDD. Where the service trenches enter the building, the trench backfill must be compacted as structural fill to a minimum of 100% SPMDD. Any trench backfill below a pavement structure should be compacted to 100% SPMDD within 1 m from the top of subgrade level. Based on the results of in-situ moisture content tests carried out on the native overburden deposits, the materials may be suitable for reuse as trench backfill. Any overly wet material may require drying prior to reusing as backfill.

7 PAVEMENT DESIGN AND CONSTRUCTION

7.1 SUBGRADE PREPARATION

Based on the borehole findings, and anticipated Site grading, the pavement subgrade for the proposed access roads and parking lots will likely consist compact sand. The subgrade comprised of native inorganic undisturbed soils or approved compacted fill materials would provide adequate support to the pavement structure of the proposed access roads and parking lots, provided a subgrade is proof rolled and approved by the geotechnical engineer prior to the construction of the pavement structure.

Where there is existing fill on the site, caution needs to be exercised in the preparation of the subgrade on this material. It is recommended that any subgrades comprising of existing fill be inspected for obvious soft/loose areas and presence of deleterious materials. Should such areas be found, these should be replaced or treated as advised by the geotechnical engineer.

The most severe loading conditions on pavement areas and the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of sub-base fills, restricted construction lanes, and half-loads during paving may be required, especially if construction is carried out during wet weather conditions.

7.2 RECOMMENDED FLEXIBLE PAVEMENT STRUCTURE

It is understood that the proposed site will include truck docks, truck and trailer storage area, as well as light duty parking lot. Details pertaining to proposed pavement structures or anticipated traffic usage were not known at the time of this report. The preliminary pavement structures in Table 7.1 are recommended based on the anticipated subgrade conditions (i.e., predominantly sands) for industrial roadways, as well as truck and trailer parking, and light-duty parking (such as employee parking).

Table 7.1 Recommended Pavement – Component Thicknesses

Pavement Layer	Compaction Requirements	Heavy Duty Pavement Design (Driveways)	Light Duty Pavement Design (Parking Areas)
Surface Course Asphaltic Concrete HL3 (OPSS 1150)	97% Maximum Relative Density (OPSS 310)	40 mm	40 mm
Base Course Asphaltic Concrete HL8 (OPSS 1150) / HDBC	97% Maximum Relative Density (OPSS 310)	90 mm (HDBC)	50 mm (HL8)
Base Course: Granular ‘A’ (19mm Crusher Run)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular B (50mm Crusher Run)		400 mm	350 mm

For truck parking, either rigid pavement should be used, or the parking should be left unpaved as flexible pavements settle under the heavy loads of wheels of trucks parked for relatively long periods of time. Concrete pavements are more resilient to settlement. Gravel structures settle but settled areas can easily be replenished by placing additional gravel and compacting it in place.

The following rigid pavement structure included in table 7.2 can be used for the dolly pads and forklift ramps.

Table 7.2 Recommended Pavement Structure for Truck Parking Area

Pavement Layer	Compaction Requirements	Concrete Pavement
Portland Cement Concrete Layer Class C-2, 32 MPa	-	180 mm
Base Course: Granular ‘A’ or 19mm Crusher Run	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm
Subbase Course: Granular B Type II		250 mm

The concrete pavement should either be designed as continuously reinforced concrete or Jointed Plain Concrete Pavement (JPCP). JPCP uses contraction joints to control cracking and does not use any reinforcing steel. Transverse joint spacing is selected such that temperature and moisture stresses do not produce intermediate cracking between joints. This typically results in a spacing no longer than about 6.1 m (20 ft.). Dowel bars are typically used at transverse joints (in the direction of traffic flow) to assist in load transfer to prevent breaking of the slab edges under repeated wheel impacts. Tie bars are typically used at longitudinal joints.

These structures should provide a typical pavement service life, provided regular maintenance is carried out during the life cycle of the pavements. The above pavement structure recommendations are based on typical expected use along with anticipated subgrade conditions. It should be noted that no design traffic data was provided to Stantec at the time of this design, and thus a detailed pavement design analysis was not carried out.

The pavement subgrade must be proof rolled under the supervision of geotechnical personnel prior to Granular ‘B’ placement to identify any soft areas where thickened subbase is warranted.

The base and subbase materials should be compacted to a minimum of 100% SPMDD. The asphaltic concrete should be compacted to a minimum of 92% of Maximum Relative Density (MRD). Asphalt compaction must be carried out as specified in OPSS 310.

It is understood that drainage ditches will be utilized for the proposed site. To achieve effective drainage, the finished pavement surface and underlying subgrade should be free of depressions and should be sloped (preferably at a minimum grade of 2%). Surface water should not be allowed to pond adjacent to outside edges of the pavement structures.

There is potential that the portions of the on-site native sand and gravel could meet gradation specifications for OPSS 1010 Granular ‘B’ if cobbles and boulders are sorted out. Additional testing would be required to confirm if the material meets the gradation requirements.

8 CLOSURE

Use of this report is subject to the Statement of General Conditions provided in **Appendix A**. It is the responsibility of Cedar Creek Road Holdings Inc. who is identified as “the Client” within the Statement of General Conditions, and its agents to review the conditions and to notify Stantec Consulting Ltd. should any of these not be satisfied. The Statement of General Conditions addresses the following:

- Use of the report.
- Basis of the report
- Standard of care.
- Interpretation of site conditions.
- Varying or unexpected site conditions; and,
- Planning, design, or construction.

This report has been prepared by Essa Nimer, reviewed by Hassan Gilani and Raid Khamis .

Respectfully Submitted,

STANTEC CONSULTING LTD.

APPENDIX A

A.1 STATEMENT OF GENERAL CONDITIONS

STATEMENT OF GENERAL CONDITIONS

USE OF THIS REPORT: This report has been prepared for the sole benefit of the Client or its agent and may not be used by any third party without the express written consent of Stantec Consulting Ltd. and the Client. Any use which a third party makes of this report is the responsibility of such third party.

BASIS OF THE REPORT: The information, opinions, and/or recommendations made in this report are in accordance with Stantec Consulting Ltd.'s present understanding of the site specific project as described by the Client. The applicability of these is restricted to the site conditions encountered at the time of the investigation or study. If the proposed site specific project differs or is modified from what is described in this report or if the site conditions are altered, this report is no longer valid unless Stantec Consulting Ltd. is requested by the Client to review and revise the report to reflect the differing or modified project specifics and/or the altered site conditions.

STANDARD OF CARE: Preparation of this report, and all associated work, was carried out in accordance with the normally accepted standard of care in the state or province of execution for the specific professional service provided to the Client. No other warranty is made.

INTERPRETATION OF SITE CONDITIONS: Soil, rock, or other material descriptions, and statements regarding their condition, made in this report are based on site conditions encountered by Stantec Consulting Ltd. at the time of the work and at the specific testing and/or sampling locations. Classifications and statements of condition have been made in accordance with normally accepted practices which are judgmental in nature; no specific description should be considered exact, but rather reflective of the anticipated material behavior. Extrapolation of in situ conditions can only be made to some limited extent beyond the sampling or test points. The extent depends on variability of the soil, rock and groundwater conditions as influenced by geological processes, construction activity, and site use.

VARYING OR UNEXPECTED CONDITIONS: Should any site or subsurface conditions be encountered that are different from those described in this report or encountered at the test locations, Stantec Consulting Ltd. must be notified immediately to assess if the varying or unexpected conditions are substantial and if reassessments of the report conclusions or recommendations are required. Stantec Consulting Ltd. will not be responsible to any party for damages incurred as a result of failing to notify Stantec Consulting Ltd. that differing site or sub-surface conditions are present upon becoming aware of such conditions.

PLANNING, DESIGN, OR CONSTRUCTION: Development or design plans and specifications should be reviewed by Stantec Consulting Ltd., sufficiently ahead of initiating the next project stage (property acquisition, tender, construction, etc), to confirm that this report completely addresses the elaborated project specifics and that the contents of this report have been properly interpreted. Specialty quality assurance services (field observations and testing) during construction are a necessary part of the evaluation of sub-subsurface conditions and site preparation works. Site work relating to the recommendations included in this report should only be carried out in the presence of a qualified geotechnical engineer; Stantec Consulting Ltd. cannot be responsible for site work carried out without being present.

APPENDIX B

B.1 SITE LOCATION PLAN

B.2 BOREHOLE LOCATIONS PLAN

NOTES
1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 17N.
2. IMAGERY: ESRI, © 2024.

200m 0 200 400m

1 : 20 000

DECEMBER 2024
Project No. 161414214

C:\Users\gbriones\appdata\local\temp\AcPublish_15052\161414214_Site Location Plan.dwg
Printed: Jan 04, 2024 By: G. Briones

Stantec

300 - 1331 Clyde Avenue
Ottawa, ON, Canada K2C 3G4
www.stantec.com

Client/Project
CEDAR CREEK ROAD HOLDINGS INC.
GEOTECHNICAL INVESTIGATION
2509 CEDAR CREEK ROAD, AYR, ON

Drawing No.
1

Title
SITE LOCATION PLAN

Stantec

100-300 Hagey Boulevard
Waterloo ON N2L 0A4
www.stantec.com

LEGEND

- ▬ · · ▬ PROPERTY BOUNDARY
- - - - LOT LINE
- ⊕ APPROXIMATE BOREHOLE WITH MONITORING WELL LOCATION
- (302.794) BOREHOLE ELEVATION (mASL)

NOTES

1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 17N.
2. LOT LINES TAKEN FROM CAD FILE 161414214_C-SP.DWG
3. IMAGERY: © 2022 MICROSOFT CORPORATION © 2022 MAXAR © CNES (2022) DISTRIBUTION AIRBUS DS.

30m 0 30 60m
1 : 3000

JANUARY 2024
Project No. 161414214

Client/Project

CEDAR CREEK ROAD HOLDINGS INC.
GEOTECHNICAL INVESTIGATION
2509 CEDAR CREEK ROAD, AYR, ONTARIO

Drawing No.
2

Title

BOREHOLE LOCATION PLAN

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2024/01/04 9:56 AM By: Briones, Gliceria

APPENDIX C

C.1 SYMBOLS AND TERMS USED ON BOREHOLES

C.2 BOREHOLE LOGS

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

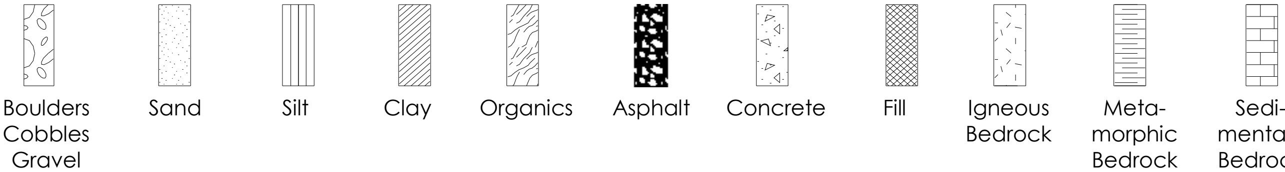
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼	measured in standpipe, piezometer, or well
▽	inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

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BOREHOLE RECORD

N: 546 033 E: 4 798 024

Sheet 1 of 1

BH/MW01-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

DATUM

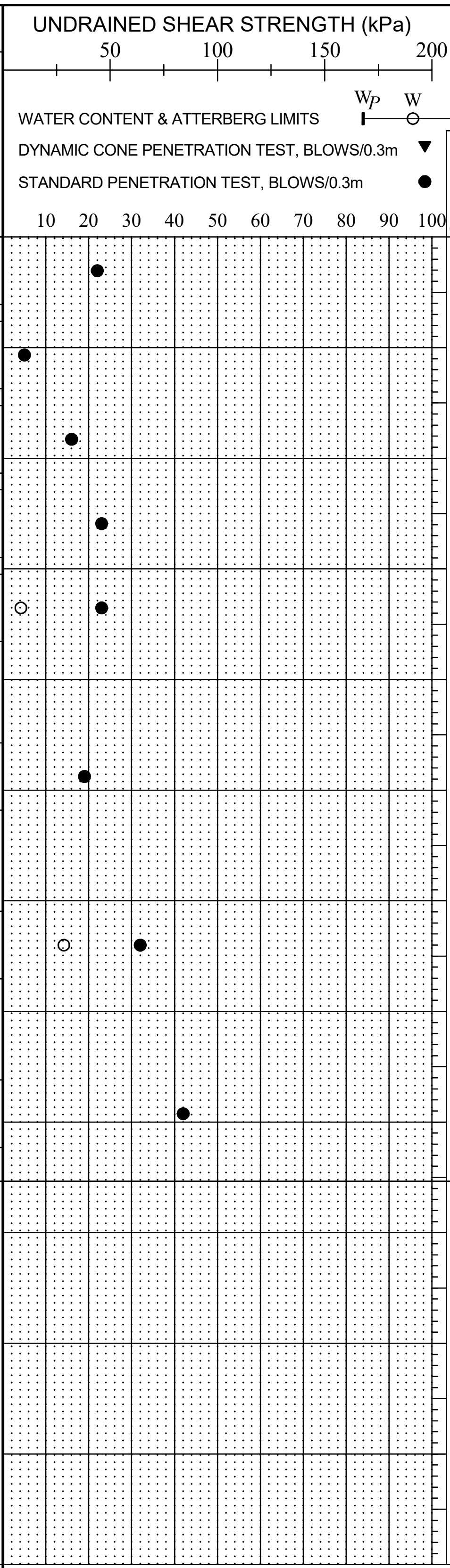
DATES: BORING February 16, 2022

WATER LEVEL

TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	**309.7**	**Snow, Farm Field**
		TOPSOIL: dark brown, silty sand - trace gravel - frozen to moist
	308.9	Loose to compact, brown, SAND (SP) - trace silt and gravel - moist - moist to wet
	304.2	Dense, brown, SAND (SP) with gravel - wet to saturated
	301.9	Dense, brown, sandy SILT (ML) - occasional boulder - saturated
	301.1	Borehole terminated at 8.5 m below existing ground due to auger refusal on boulder and sand heave. 50 mm diameter PVC well installed with 3.0 m screen between 8.5 - 5.5 m depths below grade. Backfilled with sand from 9.4 m up to 4.9 m, and bentonite from 4.9 m depths to surface. Finished with pedestal cover at surface.

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	1	360/610	22	
3	SS	2	360/610	5	
6	SS	3	460/610	16	
8	SS	4	460/610	23	
11	SS	5	480/610	23	
16	SS	6	460/610	19	
21	SS	7	610/610	32	15 78 7
26	SS	8	610/610	42	



□ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

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BOREHOLE RECORD

N: 546 169 E: 4 798 068

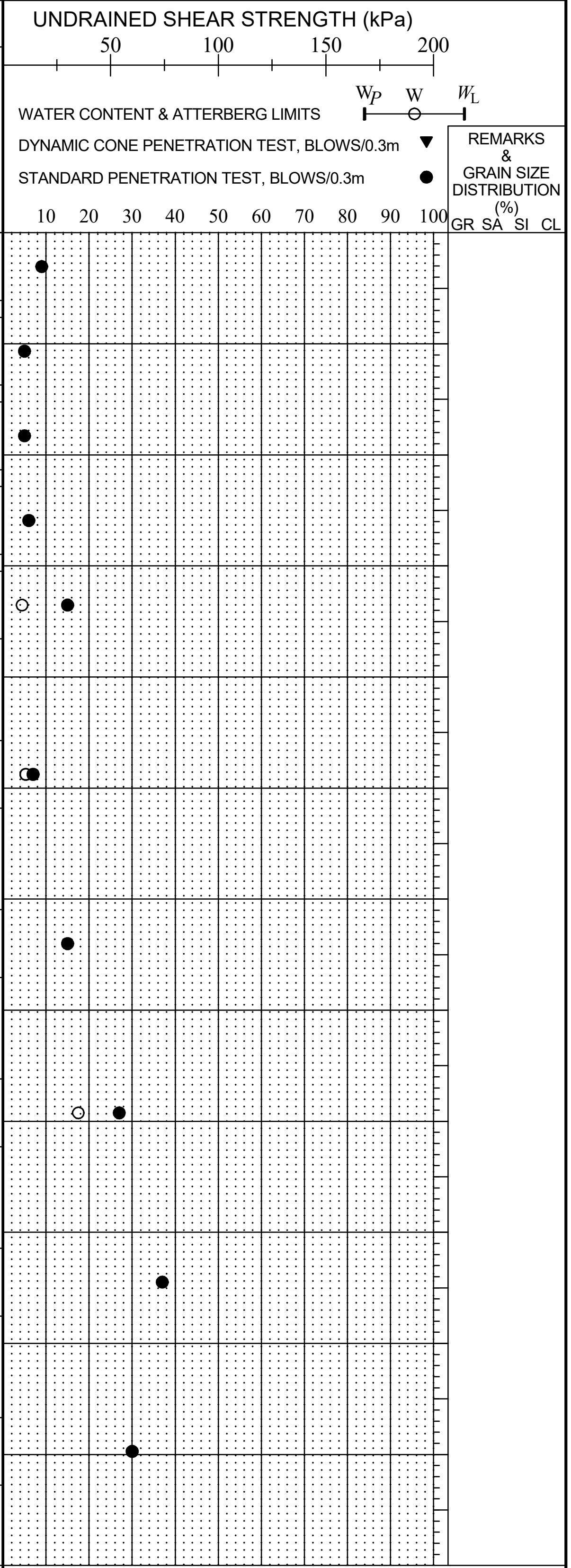
Sheet 1 of 2

BH/MW02-22

CLIENT Cedar Creek Road Holdings Inc.
PROJECT No. 161414214
LOCATION 2509 Cedar Creek Road, Ayr, Ontario
DATUM
DATES: BORING February 16, 2022
WATER LEVEL
TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	**314.9**	**Snow, Farm Field**
0 – 0.9	314.0	TOPSOIL: dark brown, sandy silt - trace clay and gravel - trace rootlets and organics - frozen to moist
0.9 – 2.3	312.6	Loose, brown to light brown, sandy SILT (ML) - some to trace clay - moist to saturated
2.3 – 7.6	307.3	Loose to compact, brown, SAND AND GRAVEL (SP-GP) - occasional cobbles - moist
7.6 – 12		Compact to dense, brown, SILT (ML) - with sand to some sand - trace to some clay - moist to saturated

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0 – 2	SS	1	560/610	9
2.5 – 4.5	SS	2	460/610	5
5 – 7	SS	3	410/610	5
7.5 – 9.5	SS	4	250/610	6
10 – 12	SS	5	360/610	15
15 – 17	SS	6	430/610	7
20 – 22	SS	7	480/610	15
25 – 27	SS	8	530/610	27
30 – 32	SS	9	580/610	37
35 – 37	SS	10	610/610	30



Continued Next Page

☐ Field Vane Test, kPa
◘ Remoulded Vane Test, kPa
△ Pocket Penetrometer Test, kPa

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BOREHOLE RECORD

N: 546 169 E: 4 798 068

Sheet 2 of 2

BH/MW02-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

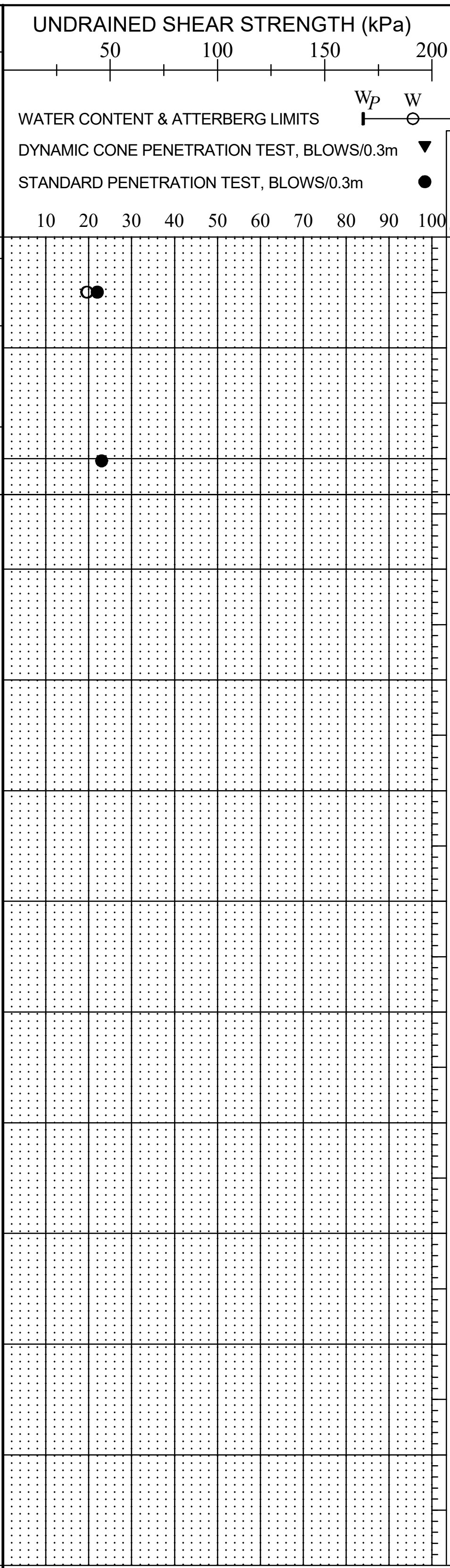
DATUM

DATES: BORING February 16, 2022

WATER LEVEL

TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION	STRATA PLOT	WATER LEVEL	DEPTH (ft)	SAMPLES TYPE	SAMPLES NUMBER	SAMPLES RECOVERY (mm) TCR(%) / SCR(%)	SAMPLES N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
12	**302.9** 302.7	Compact, brown, SILT (ML) - some to trace sand - some clay - saturated			40 41	SS	11	610/610	22	
13					42 43 44 45					
14	300.6				46	SS	12	610/610	23	
15 16 17 18 19 20 21 22 23 24		Borehole terminated at 14.3 m below existing ground. 50 mm diameter PVC well installed with 3.0 m screen between 13.7 - 10.7 m depths below grade. Backfilled with sand from 13.7 m up to 10.3 m, and bentonite from 10.3 m depths to surface. Finished with pedestal cover at surface. Well cluster tag # A311058.			47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74 75 76 77 78					



☐ Field Vane Test, kPa

▣ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

N: 546 108 E: 4 797 909

Sheet 1 of 1

BH/MW03-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

DATUM

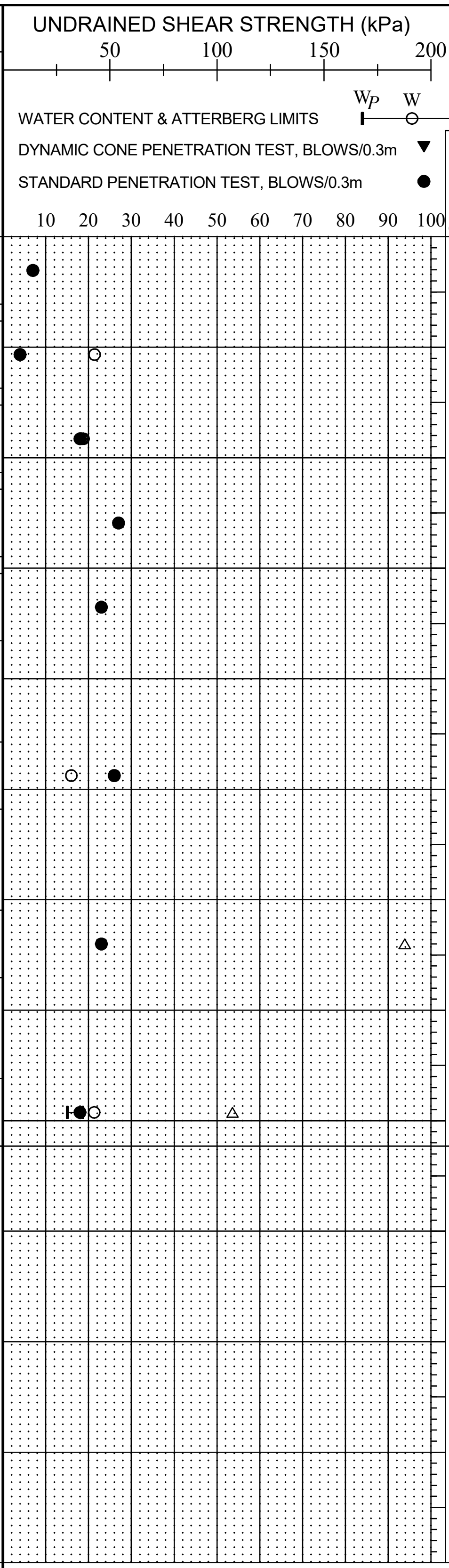
DATES: BORING February 15, 2022

WATER LEVEL

TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	**308.1**	**Snow, Farm Field**
0 – 0.7	307.4	TOPSOIL: black to dark brown, sandy silt - trace cobbles - frozen
0.7 – 1.5	306.6	Very loose to loose, brown, SAND (SP) - saturated
1.5 – 6.2	301.9	Compact, brown, SILT (ML) - sandy to trace sand - saturated, dilatant
6.2 – 8.2	299.9	Very stiff, grey brown, silty CLAY (CL-ML) - with sandy layers, saturated - APL to WTPL
8.2		Borehole terminated at 8.2 m below existing ground. 50 mm diameter PVC well installed with 3.0 m screen between 7.6 - 4.6 m depths below grade. Backfilled with sand from 7.6 m up to 4.2 m, and bentonite from 4.2 m depths to surface. Finished with pedestal cover at surface.

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
0 – 2	SS	1	380/610	7	
2.5 – 4.5	SS	2	460/610	4	
5 – 7	SS	3	480/610	18	
7.5 – 9.5	SS	4	480/610	27	
10 – 12	SS	5	530/610	23	
15 – 17	SS	6	530/610	26	0 7 86 7
20 – 22	SS	7	610/610	23	
25 – 27	SS	8	610/610	18	0 6 63 31



☐ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

BOREHOLE RECORD

N: 546 101 E: 4 797 789

Sheet 1 of 1

BH/MW04-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

DATUM

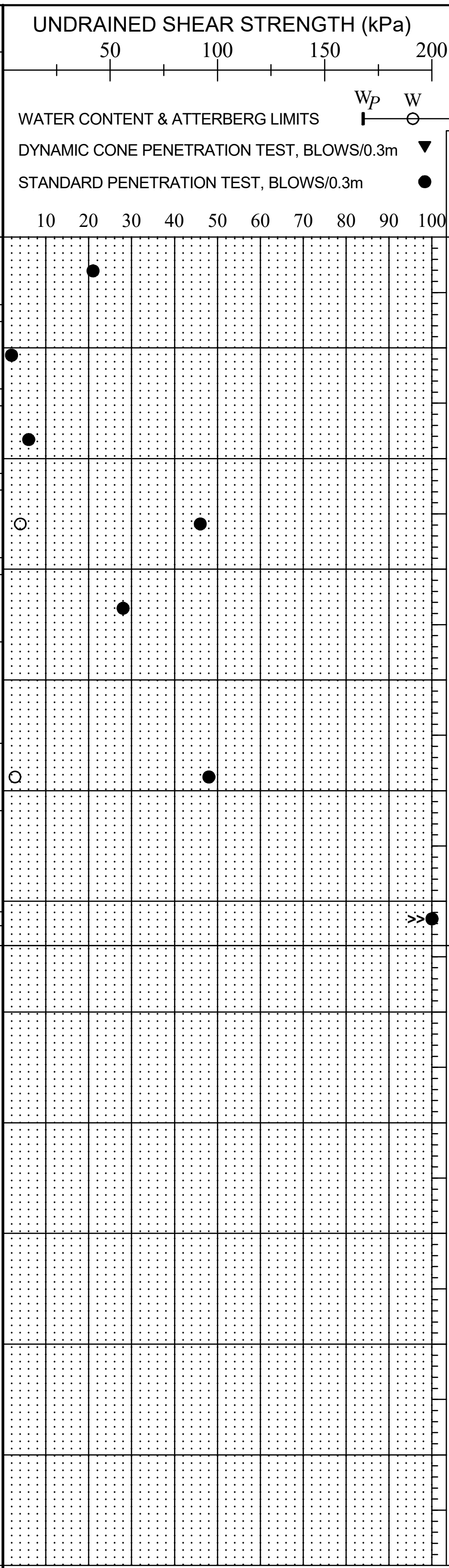
DATES: BORING February 17, 2022

WATER LEVEL

TPC ELEVATION

ELEVATION (m)	STRATA DESCRIPTION
311.8	**Snow, Farm Field**
	TOPSOIL: dark brown, silty sand - frozen to wet
310.5	Loose to very dense, brown, GRAVEL (GW-GC) with silt and sand - occasional to frequent cobbles - moist
305.4	Borehole terminated at 6.4 m below existing ground due to auger refusal on boulder. 50 mm diameter PVC well installed with 3.0 m screen between 6.0 - 3.0 m depths below grade. Backfilled with sand from 6.0 m up to 2.7 m, and bentonite from 2.7 m depths to surface. Finished with pedestal cover at surface.

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	1	560/610	21	
3	SS	2	25/610	2	
6	SS	3	150/610	6	
9	SS	4	510/610	46	
11	SS	5	460/610	28	
16	SS	6	300/610	48	51 40 9
20	SS	7	76/120	50/130	



☐ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

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BOREHOLE RECORD

N: 546 106 E: 4 797 572

Sheet 1 of 1

BH/MW05-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

DATUM

DATES: BORING February 18, 2022

WATER LEVEL

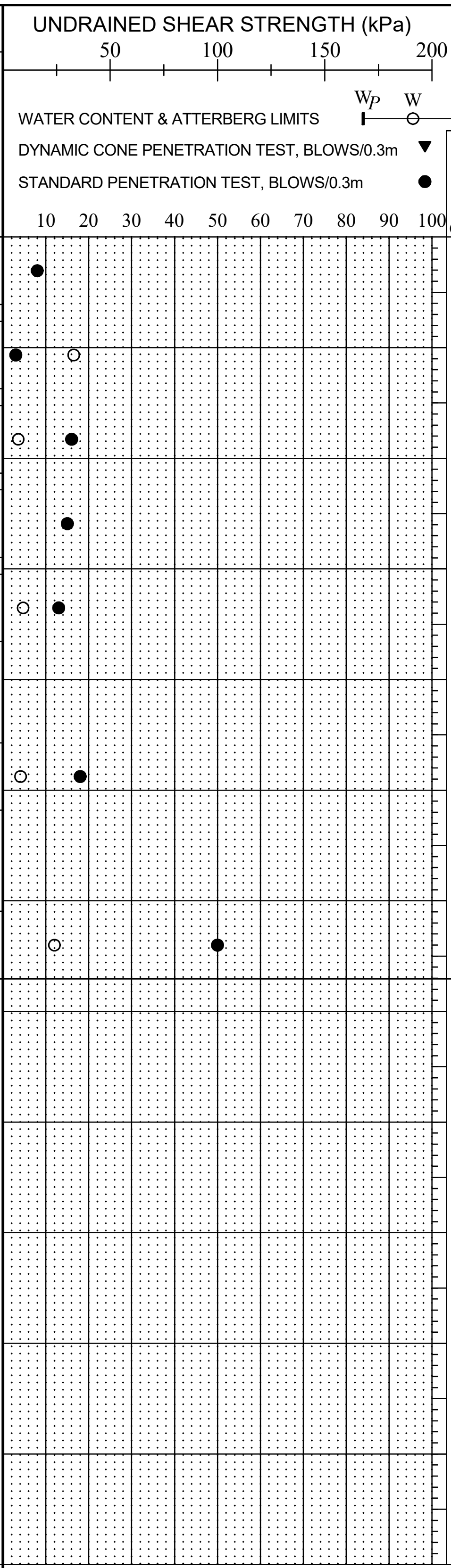
TPC ELEVATION

ELEVATION (m)	STRATA DESCRIPTION
306.7	**Snow, Farm Field**
306.0	TOPSOIL: dark brown, silty sand - trace rootlets and organics - frozen to moist
305.2	Very loose, brown, silty SAND (SM) - trace gravel - wet to moist
300.6	Compact, brown, SAND (SP) - trace silt - damp
300.0	Dense to very dense, brown, SAND (SP/SM) - silty to trace silt - trace gravel to gravelly - occasional cobble - moist to wet

Borehole terminated at 6.7 m below existing ground.

50 mm diameter PVC well installed with 3.0 m screen between 6.0 - 3.0 m depths below grade. Backfilled with sand from 6.0 m up to 2.7 m, and bentonite from 2.7 m depths to surface. Finished with pedestal cover at surface.

DEPTH (ft)	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
0 - 2	SS	1	510/610	8
2.5 - 4.5	SS	2	410/610	3
5 - 7	SS	3	510/610	16
7.5 - 9.5	SS	4	560/610	15
10 - 12	SS	5	610/610	13
15 - 17	SS	6	480/610	18
20 - 22	SS	7	610/610	50



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

☐ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

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BOREHOLE RECORD

N: 546 294 E: 4 797 689

Sheet 1 of 1

BH/MW06-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

DATUM

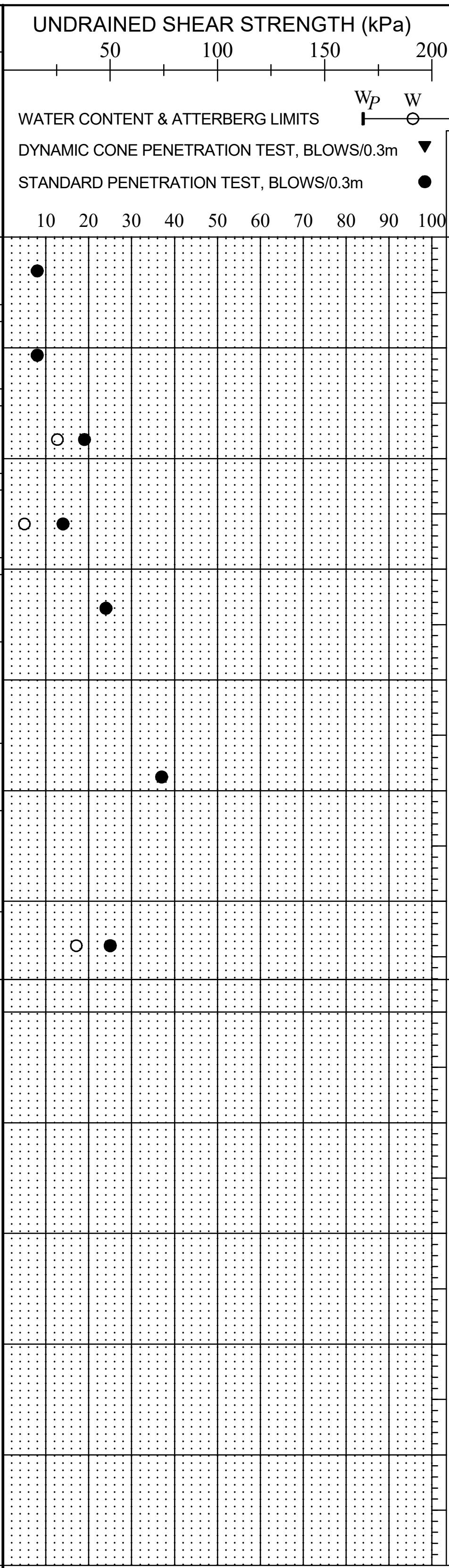
DATES: BORING February 17, 2022

WATER LEVEL

TPC ELEVATION

DEPTH (m)	ELEVATION (m)	STRATA DESCRIPTION
0	**306.2**	**Snow, Farm Field**
		TOPSOIL: dark brown, sandy silt - trace clay - frozen to wet
	305.0	Compact, brown, SAND (SP) - with silt to silty - trace gravel - moist to wet
	303.1	Compact to dense, brown, gravelly SAND (SP-SM) - some silt - moist to wet
	299.8	Dense, brown, sandy SILT (ML) - saturated, dilatant
	299.5	Borehole terminated at 6.7 m below existing ground. 50 mm diameter PVC well installed with 3.0 m screen between 6.0 - 3.0 m depths below grade. Backfilled with sand from 6.0 m up to 2.7 m, and bentonite from 2.7 m depths to surface. Finished with pedestal cover at surface.

DEPTH (ft)	SAMPLES TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
1	SS	1	410/610	8
3	SS	2	150/610	8
6	SS	3	360/610	19
8	SS	4	510/610	14
11	SS	5	510/610	24
16	SS	6	360/610	37
21	SS	7	510/610	25



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

☐ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

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BOREHOLE RECORD

N: 546 154 E: 4 797 375

Sheet 1 of 1

BH/MW07-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

DATUM

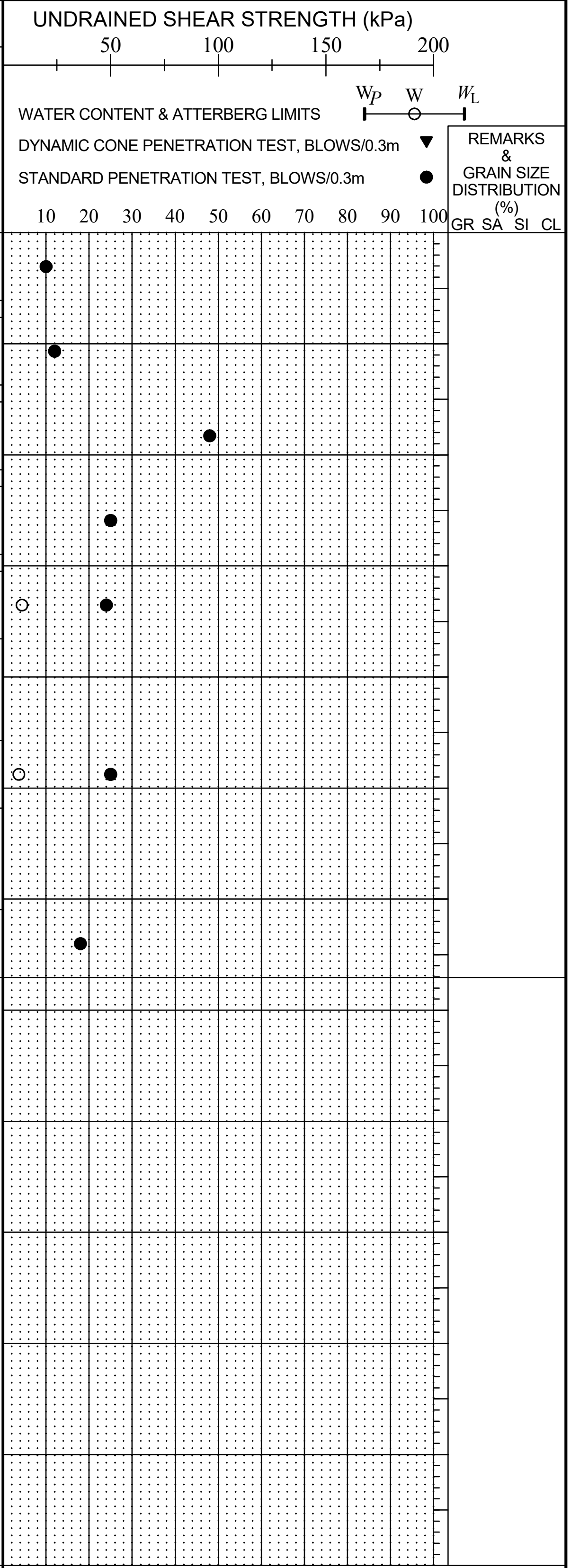
DATES: BORING February 17, 2022

WATER LEVEL

TPC ELEVATION

ELEVATION (m)	STRATA DESCRIPTION
306.4	**Snow, Farm Field**
305.6	TOPSOIL: dark brown, sandy silt - trace clay and gravel - frozen to moist
303.3	Compact to dense, brown, GRAVEL (GW-GC) with silt and sand - occasional cobbles - moist - compact
299.7	Compact, brown, SAND (SP) - trace silt and gravel - moist
	Borehole terminated at 6.7 m below existing ground. 50 mm diameter PVC well installed with 3.0 m screen between 6.0 - 3.0 m depths below grade. Backfilled with sand from 6.0 m up to 2.7 m, and bentonite from 2.7 m depths to surface. Finished with pedestal cover at surface.

TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
SS	1	610/610	10
SS	2	410/610	12
SS	3	410/610	48
SS	4	360/610	25
SS	5	460/610	24
SS	6	460/610	25
SS	7	510/610	18



☐ Field Vane Test, kPa

◘ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

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BOREHOLE RECORD

N: 546 399 E: 4 797 336

Sheet 1 of 1

BH/MW08-22

CLIENT Cedar Creek Road Holdings Inc.

PROJECT No. 161414214

LOCATION 2509 Cedar Creek Road, Ayr, Ontario

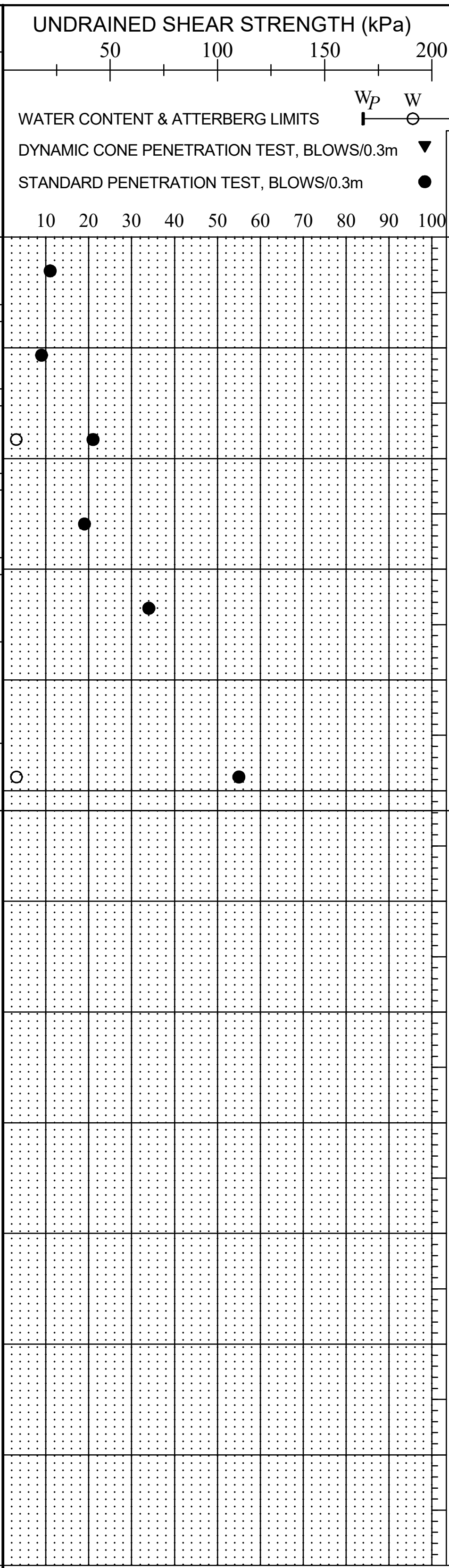
DATUM

DATES: BORING February 17, 2022

WATER LEVEL

TPC ELEVATION

ELEVATION (m)	STRATA DESCRIPTION	TYPE	NUMBER	RECOVERY (mm) TCR(%) / SCR(%)	N-VALUE OR RQD(%)
301.8	**Snow, Farm Field**				
	TOPSOIL: dark brown, sandy silt - trace clay and gravel - frozen to moist	SS	1	610/610	11
300.7	Loose to compact, brown, SAND (SP) - some gravel - trace silt - occasional cobbles - moist	SS SS SS	2 3 4	360/610 410/610 410/610	9 21 19
298.7	Dense to very dense, brown, SAND (SP) - some gravel to gravelly - trace silt - frequent cobbles - moist to damp	SS SS	5 6	460/610 360/610	34 55
296.6	Borehole terminated at 5.2 m below existing ground due to auger refusal on boulder. 50 mm diameter PVC well installed with 3.0 m screen between 5.2 - 2.2 m depths below grade. Backfilled with sand from 5.2 m up to 1.8 m, and bentonite from 1.8 m depths to surface. Finished with pedestal cover at surface.				



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

□ Field Vane Test, kPa

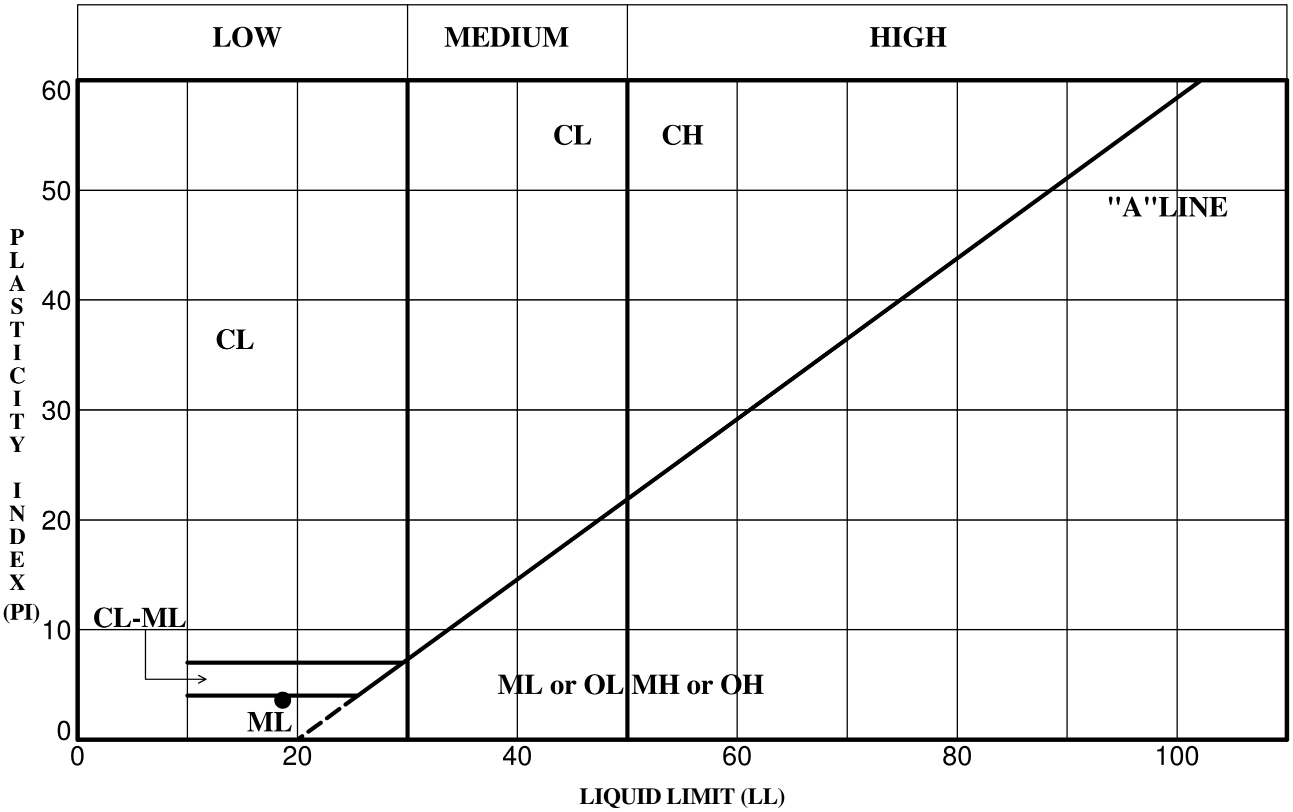
▣ Remoulded Vane Test, kPa

△ Pocket Penetrometer Test, kPa

APPENDIX D

D.1 LABORATORY TESTING RESULTS

PLASTICITY CHART



	Specimen	Depth (m)	LL	PL	PI	Fines	W%	Classification
●	BH/MW03-22	7.9	19	15	4	94	21	SILTY CLAY (CL-ML)

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Project: 2509 Cedar Creek Road

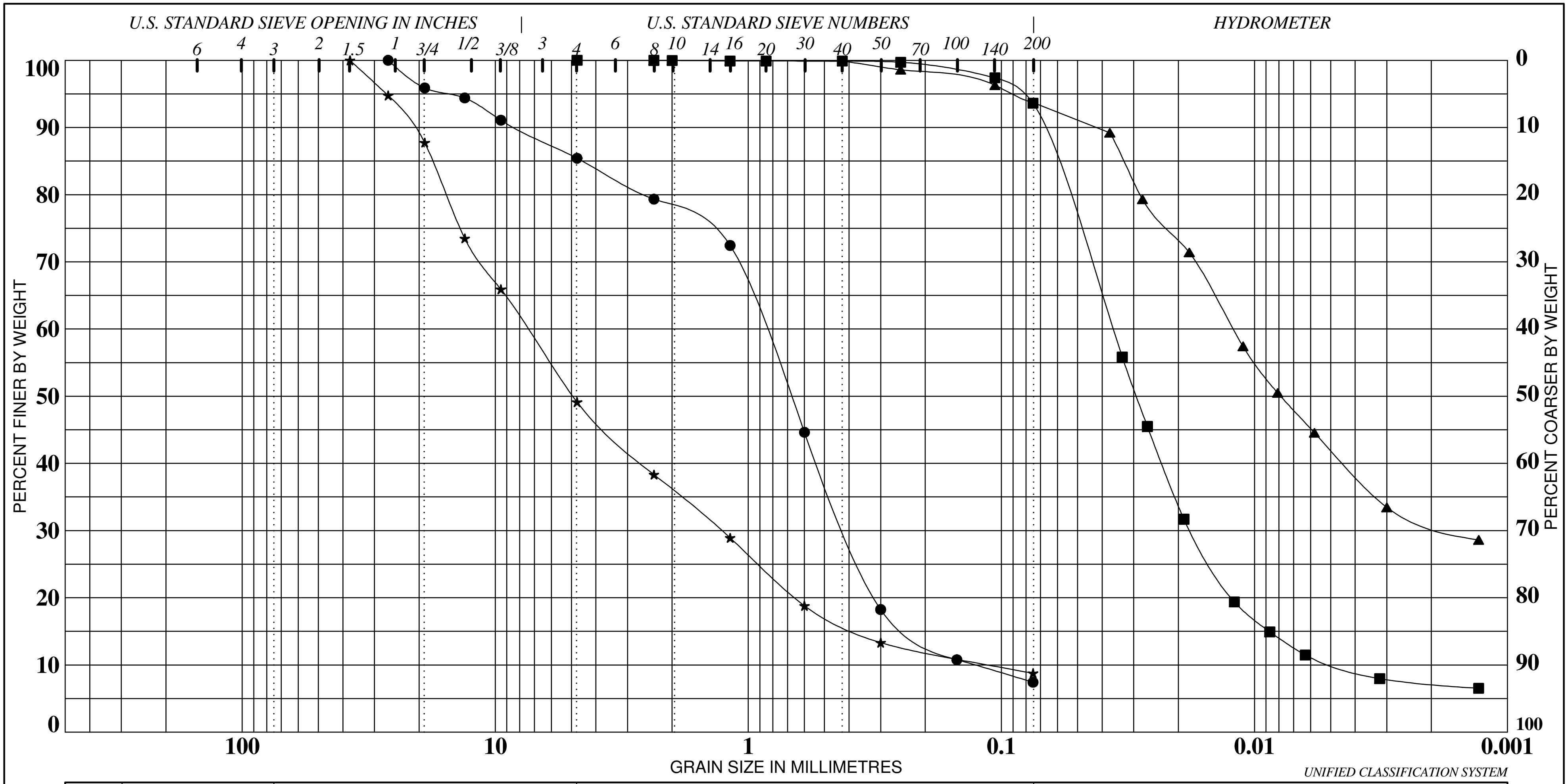
Location: 2509 Cedar Creek Road

Project No.: 161414214

ATTERBERG LIMITS
(ASTM D4318)

Figure: 1

Remarks:



BLDs	COBBLES	GRAVEL		SAND			SILT & CLAY	
		coarse	fine	coarse	medium	fine	SILT	CLAY

Sample	Depth (m)	Description	W%	W_L	W_P	I_P	%Gravel	%Sand	%Silt	%Clay
● BH/MW01-22	6.4	POORLY-GRADED SAND with GRAVEL (SP)	14				15	78	7	
■ BH/MW03-22	4.9	SILT (ML)	16				0	7	86	7
▲ BH/MW03-22	7.9	SILTY CLAY (CL-ML)	21	19	15	4	0	6	63	31
★ BH/MW04-22	4.9	WELL-GRADED GRAVEL with SILT and SAND (GW-GC)	3				51	40	9	

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Project: 2509 Cedar Creek Road

Location: 2509 Cedar Creek Road

Project No.: 161414214

GRADATION CURVE (ASTM D422)

Figure: 2

Remarks: