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Geotechnical Investigation
Proposed Development
3095 Palladium Drive
Ottawa, Ontario

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Geotechnical Investigation
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Ottawa, Ontario

June 20, 2023
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out in support of a Site Plan Control application for the proposed development of the property at 3095 Palladium Drive in Ottawa, Ontario (herein known as ‘the site’).

The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of boreholes and, based on the information obtained, to provide preliminary engineering guidelines and recommendations on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

This study was carried out in general accordance with our proposal dated May 2, 2023.

This report is subject to the ‘Conditions and Limitations of this Report’, which follows the text of the report, and which are considered an integral part of the report.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Preliminary details in the form of a site sketch with general layout details have been provided to GEMTEC. Based on the preliminary details, the following is understood/assumed:

- The proposed development will include 6 commercial buildings (ranging between about 350 and 900 square metres in area), a car wash, parking and laneway areas and service installations (i.e., watermains and sewers);
- The buildings will consist of single storey, slab-at-grade structures (i.e., no below grade levels);
- The maximum grade raise above existing ground surface at the site is proposed to be less than 1.0 metres;
- The proposed service installations (i.e., watermains and sewers) will be at a maximum depth of about 3 metres below existing ground surface; and,
- It is proposed that a subsurface stormwater collection system(s) be constructed below the proposed parking areas.

2.2 Site Description

The subject site is currently undeveloped, has an area of about 1.8 hectares, and is currently vacant, generally flat, and has some limited re-growth of vegetation following site works that are reported to have been completed in 2015 and 2016. Generally, the site is surrounded by developed (commercial) lands.

3.0 REVIEW OF AVAILABLE INFORMATION

3.1 Geological Mapping

Surficial geology maps of the Ottawa area indicate that the overburden deposits at the site consist of offshore marine sediments consisting of silt and clay. Drift thickness maps indicate that the thickness of the overburden deposits ranges from about 10 to 15 metres. Bedrock geology maps indicate that the overburden deposits are underlain by interbedded limestone and shale bedrock of the Verulam formation.

Fill associated with past site preparation works should be expected.

3.2 Previous Investigations by Others

GEMTEC has been provided with a Supplemental Geotechnical Investigation report that was prepared by Paterson Group (report No. PG3115-2, Rev 2, dated September 2, 2016). The subsurface conditions encountered in the boreholes that were advanced on the site during the investigation are noted as being native deposits of silty clay overlying silty sand/sandy silt overlying glacial till which was proven to a depth of 6.7 metres in one of the boreholes. Groundwater measurements from within the monitoring wells that were installed in the boreholes indicated that the groundwater level was at about 1.1 metres below ground surface in December of 2010 and at about 2.2 to 2.3 metres below ground surface in November of 2014.

The approximate locations of the previously advanced boreholes are shown on the Borehole Location Plan, Figure 1. The results of the previous boreholes are provided on the Record of Borehole sheets (by others) in Appendix B.

4.0 METHODOLOGY

4.1 Geotechnical Investigation

The fieldwork for this investigation was carried out on May 23, 2023, at which time four boreholes (numbered 23-01 to 23-04) were advanced at the site. The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 1.

The boreholes were advanced using a rubber-track mounted drill rig supplied and operated by CCC Geotechnical & Environmental Drilling Ltd. Of Ottawa, Ontario. Standard penetration tests were carried out where possible in the boreholes within the overburden deposits and samples of the soils encountered were recovered using drive open sampling equipment. A monitoring well was installed in two of the boreholes to facilitate measurement of the stabilized groundwater conditions.

The fieldwork was observed by a member of our engineering staff who directed the drilling operations, observed the in-situ testing, and logged the samples and boreholes.

Following the fieldwork, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Selected samples of the soil were tested for water content, grain size distribution, Atterberg limits, and shrinkage limit.

Two samples of the overburden soil from one of the boreholes was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The borehole locations were selected by GEMTEC personnel and positioned at the site based on the details of the proposed development and relative to existing site features, including the existing underground and overhead utilities. The ground surface elevations at the borehole locations were determined using a GPS instrument and referenced to geodetic vertical network CGVD28.

4.2 Infiltration Testing and Long Term Monitoring

On June 5, 2023, infiltration testing was carried out to estimate the saturated hydraulic conductivity in the vadose zone (ASTM D5126 – 90: Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in the Vadose Zone).

Hand augured test holes for the infiltration testing were advanced immediately adjacent to the borehole/monitoring well with the same numerical identification (e.g., GP23-04 at BH23-04; Figure 1). Infiltration testing could not be completed in the vicinity of BH23-02 due to dense fill material and gravel within the test hole.

Soil samples obtained from the test holes (including GP23-02) were submitted for grain size analysis and hydrometer testing. The hydraulic conductivities of the soil material represented by soil samples were estimated based on the grain size distribution results (i.e., sieve analysis) using the HydrogeoSieve XL Version 2.3.9 spreadsheet toolkit developed by the Department of Geology at the University of Kansas (April 2023). The toolkit features up to 16 empirical relationships to estimate hydraulic conductivity based on the distribution of grain size analyses. The summary datasheets from the hydraulic conductivity calculations are provided in Appendix E.

The K_{fs} values obtained from both the Guelph permeameter testing and grain size analysis were converted to infiltration rates based on the relationship between vertical hydraulic conductivity and infiltration rates presented in “Credit Valley Conservation and Toronto and Region Conservation (2010) Low Impact Stormwater Management Planning and Design Guidelines – Version 1.0” (CVC, TRCA, 2010).

5.0 SUBSURFACE CONDITIONS

5.1 General

The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation.

Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the soil classification testing are provided in Appendix C and on the Record of Borehole sheets. The results of the chemical testing on the soil sample are provided in Appendix D.

5.2 Subsurface Conditions

5.2.1 Topsoil Fill

A layer of topsoil fill was encountered at ground surface at boreholes 23-01 and 23-02. The thickness of the layer is about 200 millimetres and can generally be described as dark brown silty sand/sandy with organic material to silt to silty clay with some gravel, trace of sand, and with organic material.

5.2.2 Fill Material

A layer of fill was encountered below the topsoil fill at boreholes 23-01 and 23-02 and at ground surface at boreholes 23-03 and 23-04. The fill extends to depths ranging from about 0.8 to 1.1 metres below ground surface (i.e., elevations 102.7 to 103.0 metres).

At borehole 23-04, a thin layer of possible topsoil was observed at the bottom of the fill layer, however for the purposes of this report it is considered to form part of the fill layer.

5.2.3 Silty Clay

Native deposits of silty clay were encountered below the fill material at all the boreholes.

At boreholes 23-01 and 23-04, the full thickness of the silty clay encountered has been weathered to a grey brown crust and the deposit extends to depths of 1.8 and 3.2 metres, respectively (i.e., elevations 102.1 and 100.6 metres).

At boreholes 23-02 and 23-03, the deposit transitions from the weather crust to a weathered deposit of layered, grey brown silty clay and silty sand which extends to depths of 3.1 and 4.0 metres below ground surface, respectively (i.e., elevations 100.7 and 99.6 metres).

Standard penetration tests carried out in the weathered clays gave N values of between about 2 and 11 blows per 0.3 metres of penetration, which reflect a stiff to very stiff consistency. Silty clay with a blow count of greater than about 2 will generally have a shear strength of greater than 100 kilopascals, which is not measurable using a standard MTO N-vane. Therefore, it is considered acceptable to assume a “stiff to very stiff” consistency in the weathered clays.

Grain size distribution testing was undertaken on 1 sample of the weathered silty clay crust. The results are provided in Appendix C and are summarized in Table 5.1.

Table 5.1 – Summary of Grain Size Distribution Test (Weathered Silty Clay)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
23-03	3	1.5 – 2.1	0	1	40	59

The results of Atterberg limit testing carried out on one sample of the weathered silty clay crust are provided on the Plasticity Chart in Appendix C and are summarized in Table 5.2. The Atterberg limit testing was carried out in general conformance with ASTM D4318.

Table 5.2 – Summary of Atterberg Limit Testing (Weathered Silty Clay)

Borehole	Sample Number	Sample Depth (metres)	Water Content (%)	LL (%)	PL (%)	PI (%)
23-03	3	1.5 – 2.1	42	48	20	28

The measured water content of one sample of the weathered silty clay crust (see Table 5.2 above) is 42 percent. The water content testing was carried out in general conformance with ASTM D4959.

A sample of the weathered crust was tested in our laboratory to asess the shrinkage limits of the silty clay at the site. The testing was performed in general accordance with ASTM D4943 (which was discontinued in 2017 by the ASTM Sponsoring Committee responsible for the standard). The modified plasticity index (PI_m) was also calculated for samples of the clay using the following formula and the results of the Atterberg limits and grain size distribution testing described previously:

$$PI_m = PI \text{ x } (\text{\% passing the 425 micrometre sieve} / 100).$$

A summary of the test and calculation results is provided in Table 5.3.

Table 5.3 – Summary of Modified Plasticity Index

Borehole ID / Sample No.	Shrinkage Limit (%)	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index (%)	Modified Plasticity Index (%)
23-03 / 3	13.5	20	48	28	28

5.2.4 Silt

At borehole 23-02 and 23-03, below the noted layered deposit, and at borehole 23-04, below the weathered crust, a transition from the silty clay to a grey silt with some clay and trace of sand and gravel occurs. This layer extends to depths between about 4.0 and 4.8 metres below ground surface (i.e., elevations 99.8 and 98.9 metres).

Standard penetration tests carried out in the silt deposits gave N values of between about 5 and 14 blows per 0.3 metres of penetration, which indicates a loose to compact relative density.

Grain size distribution testing was undertaken on 2 samples of the silt. The results are provided in Appendix C and are summarized in Table 5.4.

Table 5.4 – Summary of Grain Size Distribution Test (Silt)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
23-02	5	3.1 – 3.7	0	4	86	10
23-04	5	3.1 – 3.7	0	3	88	10

The measured water content of two samples of the silt is about 20 percent. The water content testing was carried out in general conformance with ASTM D4959.

5.2.5 Silty Sand

Native deposits of silt and sand were encountered below the weathered crust in borehole 23-01 and below the silt deposits in boreholes 23-02, 23-03 and 23-04. The content of silt and sand in the deposits varies between borehole locations and also throughout the depth of each borehole. In general, the deposits vary between layers of silty sand and sandy silt. Furthermore, the deposit is noted as possibly containing occasional sandy silt seams with varying frequency and thickness. For simplification of reporting, the deposit is herein generally referred to as silty sand.

The silty sand deposits were fully penetrated in boreholes 23-02 and 23-04, where the deposits extend to depths of 5.5 and 5.8 metres below ground surface, respectively (i.e., elevations 98.2 and 98.0 metres). Augering in boreholes 23-01 and 23-03 was terminated in the silty sand deposit at depths of 4.4 and 6.1 metres below ground surface, respectively (i.e., elevations 99.4 and 97.5 metres). Termination at borehole 23-01 was due to refusal to further advancement of the auger. Borehole 23-03 was extended using dynamic cone penetration testing (see following section).

Standard penetration tests carried out in the silty sand deposits gave N values ranging between 4 to 12 blows per 0.3 metres of penetration, which indicates a loose to compact relative density.

5.2.6 Glacial Till

Native deposits of glacial till were encountered below the silty sand at boreholes 23-02 and 23-04, at depths of about 5.5 and 5.8 metres below ground surface, respectively (i.e., elevations 98.2 and 98.0 metres). Both boreholes were terminated within the glacial till at depths of 5.8 and 6.1 metres from ground surface, respectively (i.e. elevations 98.0 and 97.7 metres), with termination at borehole 23-02 due to refusal to further advancement of the auger.

As indicated, borehole 23-03 was extended from within the silt deposits using dynamic cone penetration testing. The results of this testing, being a rise in the number of blows per 0.3 metres of penetration, suggest a possible transition in deposit type at a depth of about 6.4 metres below ground surface (i.e., elevation 97.2 metres), and which could be indicative of the interface between the silt deposit and the glacial till which was encountered in the other, noted boreholes. The dynamic cone penetration testing was terminated at a depth of about 8.1 metres below ground surface (i.e., elevation 95.5 metres) due to refusal to further advancement of the cone.

Glacial till is generally considered to be a heterogeneous mixture of all grain sizes, and often containing cobbles and boulders. At this site, the deposit can generally be described as silty sand/sandy silt with some gravel and clay. Cobbles and boulders should also be expected in the glacial till deposit.

5.2.7 Groundwater Levels

A monitoring well was installed in each of boreholes 23-02 and 23-04 to measure stabilized groundwater conditions. A round of water levels was also conducted on June 5, 2023, at which time a long-term monitoring logger was also installed at MW23-02. Table 5.5 summarizes the groundwater levels observed on May 26, 2023 and June 2, 2023.

Table 5.5 – Summary of Groundwater Levels

Date	Borehole	Well Screen	Ground Surface Elevation (metres)	Groundwater Depth (metres)	Groundwater Elevation (metres)
May 26	23-02	Silty Sand	103.8	2.3	101.5
	23-04	Silty Sand	103.8	2.3	101.6
June 2	23-02	Silty Sand	103.8	2.5	101.3
	23-04	Silty Sand	103.8	2.4	101.4

It should be noted that the groundwater levels may be higher during wet periods of the year such as the early spring or following periods of precipitation. Furthermore, as noted in the Paterson

Group test hole logs (see Appendix B), groundwater was measured at 1.1 metres below ground surface in December of 2010 (elevation information is not provided on the logs).

5.3 Soil Chemistry Relating to Corrosion

The results of chemical testing on a sample of the silty clay (weathered crust) recovered from borehole 23-02 and a sample of the layered silty clay and silty sand from borehole 23-03 are provided in Appendix B and are summarized in Table 5.6 below.

Table 5.6 – Summary of Corrosion Testing – Glacial Till

Parameter	Borehole 23-02, Sample 3 Weathered Crust	Borehole 23-03, Sample 5 Silty Clay/Silty Sand
Resistivity (Ohm.m)	60.4	47.8
Conductivity (µS/cm)	166	209
pH	7.28	7.43
Sulphate Content (µg/g)	72	123
Chloride Content (µg/g)	16	14

5.4 Infiltration Test Results

5.4.1 Grain Size

The soil samples and corresponding relationships that met the suitability criteria, based on their grain size relationships, are summarized in Table 5.7 (refer to data output in Appendix E).

Table 5.7 – Grain Size – Estimated Infiltration Rates

Location	Sampling Interval (m)	Soil Description	Hydraulic Conductivity Estimate[1] (m/s)	Estimated Infiltration based on grain size[2] (mm/hr)
GP23-02	0.75 – 0.90	Silty Clay	Geomean: 6 x 10^{-9} Range: 2 x 10^{-10} to 6 x 10^{-7}	Geomean: 5 Range: 12 to 41
GP23-04	0.75 – 0.90	Silty Clay	Geomean: 6 x 10^{-9} Range: 3 x 10^{-10} to 6 x 10^{-7}	Geomean: 5 Range: 12 to 41
BH23-02 SA5	3.1 – 3.7	Silt	Geomean: 4 x 10^{-7} Range: 2 x 10^{-8} to 1 x 10^{-5}	Geomean: 37 Range: 15 to 88

Location	Sampling Interval (m)	Soil Description	Hydraulic Conductivity Estimate[1] (m/s)	Estimated Infiltration based on grain size[2] (mm/hr)
BH23-03 SA3	1.5 – 2.1	Silty Clay	Geomean: 4 x 10^{-7} Range: 2 x 10^{-8} to 1 x 10^{-5}	Geomean: 37 Range: 15 to 87
BH23-04 SA5	3.1 – 3.7	Silt	Geomean: 3 x 10^{-7} Range: 1 x 10^{-8} to 8 x 10^{-6}	Geomean: 35 Range: 14 to 82

Notes:
1. Hydraulic conductivity estimated based on grain size distribution.
2. Infiltration based on the approximate relationship between infiltration rate and hydraulic conductivity (CVC; TRCA, 2010).

It should be noted that the estimated infiltration rates are based on soil texture only and do not consider site specific factors that may affect the infiltration rate, such as soil heterogeneity, compaction, groundwater level, etc.

5.4.2 Guelph Permeameter

The infiltration rate at one hand auger location was estimated based on in-situ testing completed using a Guelph Permeameter. The measured field saturated hydraulic conductivity (K_{fs}) was 1.0 x 10^{-8} (see Appendix E). The corresponding estimated infiltration rate, based on K_{fs}, is 13 mm/hr (Table 5.8).

Table 5.8 – Guelph Permeameter – Estimated Infiltration Rates

Location	Soil Description	Hydraulic Conductivity Field Estimate (m/s)	Estimated Infiltration Field Measured[1] (mm/hr)
GP23-04	Silty Clay	1 x 10^{-8}	14

Notes:
1. Infiltration based on the approximate relationship between infiltration rate and hydraulic conductivity (CVC; TRCA, 2010).

The estimated infiltration rate based on in-situ testing using the Guelph Permeameter, at a depth of 0.90 metres below ground surface, is 14 mm/hr in silty clay (GP23-04). In-situ Guelph Permeameter testing was not completed at shallower depths due to the presence of fill material or at greater depths, due to measured groundwater levels at 2.3 to 2.5 metres below ground surface. Higher infiltration rates are expected below the silty clay, within the silt and silty sand to sandy silt soils encountered at depths ranging from 1.78 to 3.99 metres below ground surface.

6.0 RECOMMENDATIONS

6.1 Grade Raise Restrictions

In consideration of the existing site conditions and the proposed development, the proposed grade raise at the site is assumed to be less than 1.0 metre. Furthermore, the site is underlain by deposits of stiff to very stiff silty clay (weathered crust) overlying silt, silty sand, and glacial till. As such, it is GEMTEC's opinion that a grade raise restriction is unnecessary at this site, from a geotechnical perspective.

6.2 Seismic Design of Proposed Addition

It is anticipated that the foundations of the proposed buildings will be supported on native deposits of stiff to very stiff weathered silty clay crust, silty sand, and/or glacial till, or on a pad of engineered fill constructed on the noted overburden materials.

Based on Table 4.1.8.4.A. of the National Building Code of Canada, the seismic site class can be determined based on the Average Standard Penetration Resistance or the Soil Undrained Shear Strength from the borehole data. Based on the results of this investigation, it is our opinion that Site Class D may be used for the seismic design of the structures.

In GEMTEC's opinion, the soils at this site will not be susceptible to liquefaction under the design earthquake loading due to the clay content (which is greater than 5% and more generally greater than 10%).

6.3 Excavation

The excavations for the proposed structures will be carried out through the topsoil, native deposits of silty clay and silty sand, and possibly into the glacial till. These soils are anticipated to be readily excavatable using conventional hydraulic excavation equipment in general. Boulders should be anticipated in the glacial till. As such, an allowance should be made for removal of boulders during excavation, which may require use of larger excavation equipment and slower excavation progress. Additional material may be required to fill the voids left by boulder removal below the founding levels.

The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the native silty clay would considered a Type 2 soil and the sandy silt, and the glacial till, when above the groundwater level, can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, extending upwards from the bottom of the excavation.

In the instances where the excavation though these materials will extend below the groundwater level, these soils would be classified as Type 4 soils, and therefore allowance should be made for excavation side slopes of 3 horizontal to 1 vertical, or flatter, extending upwards from the bottom of the excavation. Due to the likely variability in the subsurface conditions, inspection of the soils encountered at the time of excavation should be carried out to verify if Type 3 or Type 4 conditions are applicable.

To prevent undermining of the foundations of any existing structures (current or future), the proposed excavations should not encroach within a line extending downwards and outwards from the nearest edge of the existing foundations, at the founding level, at an inclination of 1 vertical to 1 horizontal.

The native soil deposits are very sensitive to disturbance from ponded water and construction traffic. Some disturbance and loosening of the subgrade materials could occur and allowance should be made for subexcavation, as discussed further in the following sections of this report.

Depending on the depth of the excavation, in order to reduce subgrade disturbance, allowance could also be made for a 50 to 75-millimetre-thick mud mat of low strength concrete. The mud mat should be placed over the sensitive subgrade surface immediately after exposure and inspection.

6.3.1 Temporary Shoring

Based on the project details that have been provided to GEMTEC at the time of reporting, it is assumed that shoring systems will not be required. However, upon request, GEMTEC can provide further discussion on suitable type of shoring systems if and where insufficient space is available to carry out excavations using the open excavation methods discussed in the preceding section.

6.4 Groundwater Management

Based on our previous experience, groundwater inflow from the silty clay deposits into the excavations should be relatively small and controlled during construction by pumping from filtered sumps within the excavations. However, greater groundwater inflows should be expected where silty sand and till are encountered. Where groundwater pumping is required, suitable detention and filtration will be required before discharging water. The contractor should be required to submit an excavation and groundwater management plan for review. The discharge of pumped water must be carried-out following City of Ottawa ‘Sewer Use By-Law 2003-514’. It is not expected that short term pumping during excavation will have any significant effect on nearby structures and services.

The groundwater level in May of 2023 was measured in boreholes 23-02 and 23.04 at about 2.3 metres below ground surface (elevations 101.5 and 101.6 metres). The noted level may not represent the seasonal high groundwater level, nor future conditions, as the groundwater level will fluctuate seasonally and during periods of notable precipitation, as well as possibly due to construction activities in the area of the project. As noted, groundwater was measured by

Paterson Group, during the previous investigation, at 1.1 metres below ground surface in December of 2010.

The amount of water entering the excavation for the construction of the foundations and municipal services (e.g., water, sanitary sewer, storm sewer) at this site will depend on the size and depth of the excavation, as well as the water table height. Depending on inflow volumes, dewatering permits may be required. An Environmental Activity and Sector Registry (EASR) is required for groundwater takings between 50,000 to 400,000 litres/day, and a Category 3 Permit to Take Water (PTTW) is required for water takings great than 400,000 litres/day. Based on the encountered conditions, groundwater levels and proposed excavation depths, the daily groundwater taking during construction may exceed 50,000 litres per day and, as such, an EASR may be required. EASR registration requires a Water Taking and Discharge Plan, to be completed by a Qualified Professional.

6.5 Low Impact Development (LID) Features

The boreholes advanced on-site indicate the site is underlain by relatively low permeability silty clay soils and in-situ testing completed on-site indicates infiltration rates of 14 mm/hr in silty clay (GP23-04). It is noted that higher permeability silt and silty sand to sandy silt were encountered at depths ranging from 1.8 to 4.0 metres below ground surface. The low permeability soils have limited infiltration potential and it is recommended that the LID inverts extend below the silty clay layers or that the silty clay is excavated and backfilled with higher permeability soils.

The estimated infiltration rates do not include a design safety factor. The safety correction factor depends on the ratio of mean measured infiltration rates (geometric mean measured infiltration rate at the proposed bottom elevation divided by the geometric mean infiltration rate of the least permeable soil horizon within 1.5 metres below the proposed bottom elevation). Given the higher permeability soils encountered below the silty clay, the minimum safety factor of 2.5 may be appropriate for LID features with inverts below the silty clay, which should be confirmed by the LID designer. However, the near surface low permeability soils combined with high groundwater levels ranging from 2.3 to 2.5 metres below ground surface may limit the use of LID features.

A minimum separation distance of 1.0 metre from the groundwater and proposed bottom of the proposed LID is recommended. Groundwater levels were measured at depths of 2.3 to 2.5 metres below ground surface in May and June 2023. Prior to finalizing LID system design, seasonal water level data should be obtained over a one-year period.

6.6 Foundation Design

6.6.1 Proposed Buildings

Based on the results of the investigation, the proposed buildings can be founded on shallow foundations bearing on or within the native undisturbed silty clay or silty sand or on a pad of compacted engineered fill overlying these materials.

The topsoil is considered to be highly compressible and should be removed from below the proposed foundations and floor slabs.

In areas where the proposed founding level is above the level of the native soil, or where subexcavation of disturbed material or fill is required below proposed founding level, the grade could be raised with compacted granular material (engineered fill). The engineered fill should consist of granular material meeting Ontario Provincial Standard Specifications (OPSS) requirements for Granular A or Granular B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density. To provide adequate spread of load beneath the footings, the engineered fill should extend horizontally at least 0.3 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter; the excavations should be sized for this allowance. The native soils at the site are not suitable for reuse as engineered fill for structures. Where groundwater flow is encountered, the excavation will need to be dewatered during placement of the engineered fill.

For design purposes, the foundations of the proposed buildings should be sized using the bearing values provided in Table 6.1.

Table 6.1 – Foundation Bearing Values (Proposed Buildings)

Subgrade Material	Net Geotechnical Reaction at Serviceability Limit State (kilopascals)	Factored Net Geotechnical Resistance at Ultimate Limit State (kilopascals)
Native Silty Clay/Silty Sand	100	250
Compacted Engineered Fill (overlying native silty clay and/or silty sand)	150	300

1. See section on use of insulation below foundations/structures as it may affect the allowable bearing values.

The post construction total and differential settlement of the footings at SLS should be less than 25 and 20 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

To reduce the potential for cracking in the footings, foundation walls, and concrete slabs on grade, where the footings transition between different subgrade materials, the structures could potentially be reinforced within the transition areas, as recommended by the structural engineer.

6.7 Frost Protection

6.7.1 Building Foundations

All exterior footings, adjacent to heated areas of the buildings, should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings, such as for piers, should be provided with at least 1.8 metres of earth cover for frost protection purposes. If the foundation walls/floor slabs are insulated in a manner that will reduce heat transfer (loss) to the surrounding soil at founding level, the frost protection requirements for exterior footings should then conform to that required for foundations for an unheated space/isolated footings.

Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. Where insulation will be placed below the foundations, the provided bearing pressure values for foundations may require adjustment (see below).

The type of insulation used below the foundations will depend on the stresses to be imposed on the insulation. The stress on the insulation should not exceed about 35 percent of the insulation's quoted compressive strength due to the time dependant creep characteristics of this material. The allowable stress levels for several strengths of insulation are provided in Table 6.2. Other equivalent insulation types such as Foamular C-300, 400, 600, and 1000, or expanded EPS products such as StyroRail could also be considered.

Table 6.2 – Allowable Stress Levels – Expanded Polystyrene Insulation

Insulation Type	Maximum Allowable Stress (Kilopascals)
Dow SM (or equivalent)	70
Dow Highload 40 (or equivalent)	95
Dow Highload 60 (or equivalent)	145

6.7.2 Slabs on Grade

It is assumed that the proposed buildings will be continuously heated and as such the use of insulation below the floor slabs (slabs on grade) is not considered necessary (see Section 6.9 for further information).

6.8 Foundation Backfill and Drainage

6.8.1 Foundation Walls

To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting the requirements of OPSS Granular A or Granular B Type I or II.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value. Light walk behind compaction equipment should be used next to foundation walls, and the backfill on all sides of the foundation walls should be placed concurrently to avoid excessive compaction induced stress on the foundation walls.

Where areas of proposed hard surfacing (concrete, sidewalks, pavement, etc.) will abut the proposed structures, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible fill material to reduce the effects of differential frost heaving. To reduce the magnitude of differential frost induced heaving of the hard surfacing, it is suggested that granular frost tapers be constructed from 1.5 metres below finished grade to the underside of the granular subbase material for the hard surfaced areas. The frost tapers should be sloped at 3 horizontal to 1 vertical, or flatter.

The native deposits at this site are frost susceptible, however could be considered for foundation wall backfill purposes in soft-landscaped areas (where movement and settlement of the backfill will be tolerable) provided that a suitable bond break is applied to the surface of the foundations to prevent frost jacking. A suitable bond break could consist of at least 2 layers of 6 MIL polyethylene sheeting or a proprietary plastic drainage medium (e.g. System Platon). It is also pointed out that the native soils at this site can be impacted by changes in moisture content and this could affect the ability to compact this material to the required density.

Perimeter foundation drainage is not considered necessary for a slab on grade structure at this site, provided that the floor slab level is above the finished exterior ground surface level.

6.8.2 Isolated Piers

The backfill against isolated (unheated) walls or piers should consist of free draining, non-frost susceptible material, such as OPSS Granular B Type I or II requirements. The backfill material should be placed and compacted as described in the preceding section. Other measures to prevent frost jacking of these foundation elements could be provided, if desired.

6.9 Slab on Grade Support

To provide predictable settlement performance of the slabs at grade (floor slabs), all topsoil should be removed from the area of the proposed slabs to expose the native soil deposits. Loose, disturbed, or other organic material should also be removed. The subgrade surface should then be proof rolled with a steel drum roller, under dry conditions only, and under the supervision of geotechnical personnel. Any soft areas that are evident from the proof rolling should be subexcavated and replaced with imported, compacted sand or sand and gravel meeting OPSS requirements for Granular B Type I or II.

Any necessary grade raise fill below the slab should be carried out using engineered fill consisting of compacted OPSS Granular A or B Type II (with granular B Type II being the preferred material in wetter conditions). The granular base for the proposed slab on grade should consist of at least 150 millimetres of compacted OPSS Granular A. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular A and Granular B Type II material. Since the source of recycled material cannot be determined or controlled, it is suggested that these imported granular materials be composed of 100 percent crushed rock only.

Where groundwater inflow is encountered, pumping should be carried out from sumps within the excavation during placement of the engineered fill. Adjustment to the thickness of the first lift of imported material, and the use of static rolling during compaction, may be required under wetter conditions. This should be determined in consultation with, and under observation by, a qualified geotechnical professional.

All granular material should be compacted in maximum 300 millimetre thick lifts and to at least 98 percent of the standard Proctor maximum dry density value for the granular base (OPSS Granular A) and 98 percent for the engineered fill (Granular A or OPSS Granular B Type II), using suitably sized, vibratory compaction equipment. The proximity of existing structure(s) should be considered when selecting the type of compaction equipment.

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

If any areas of the building are to remain unheated during the winter period, thermal protection of the slab on grade may be required. The design depth of frost penetration for unheated structures is 1.8 metres below ground surface. An allowance should be made for providing thermal protection in the event that frost susceptible soils will remain present below the slab and within 1.8 metres depth from slab surface. The use of polystyrene insulation could be considered and details on the insulation requirements can be provided by GEMTEC, upon request.

6.10 Site Services

6.10.1 Overview

Details regarding the proposed site servicing (watermains and sewers) were not provided to GEMTEC at the time of reporting. The following recommendations should be considered as preliminary and, upon availability, final design details for the service installations should be provided to GEMTEC for review.

It is assumed that the site services will likely be constructed using open cut methods. As previously indicated, the assumed maximum depth of excavation for the services is 3 metres below existing ground surface.

6.10.2 Excavation

The overburden excavations for the site services will be carried out through topsoil, silty clay, silty sand, and possibly into the glacial till. As well, the excavations will likely extend to below the level of the groundwater.

In the overburden, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 and for rigid service pipes in accordance with OPSD 802.031, for Type 3 or 4 soil as applicable.

The sides of the excavations within overburden soils should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the sandy soils at this site can be classified as Type 3 soils and the clay soils as Type 2. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes. For excavations below the groundwater level, the sandy soils are classified as Type 4 (according to OHSA) and allowance should be made for 3 horizontal to 1 vertical, or flatter, excavation slopes. As an alternative or where space constraints dictate, the service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

Excavation of the native overburden deposits above the groundwater level should not present significant constraints. Below the groundwater level, sloughing of the deposits into the excavation should be anticipated along with disturbance to the soils in the bottom of the excavation (e.g. basal instability). Sloughing of the excavation side slopes below the groundwater level could be reduced, where necessary, by advancing thick steel plates along the sides and front of the trench box to below the level of the excavation in combination with pumping from within the excavation.

Saturated deposits of silty clay and/or silty sand will likely be encountered at subgrade level along the proposed service alignments. These deposits are susceptible to weakening under vibration and/or repeated loading and it is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket. We recommend that a contingency allowance be made in the contract for a 300 millimetre thick subbedding layer of OPSS Granular B Type II granular material in the event that the subgrade soils are disturbed during construction.

Where the depth of the excavations will be limited to a maximum depth of 4 metres below ground surface, the excavations will have an adequate factor of safety (i.e., greater than 2) against basal instability.

6.10.3 Groundwater Management

See Section 6.4 on groundwater management.

6.10.4 Sub-Bedding for Services

In order to mitigate the risk of post construction settlement of the services, all disturbed material should be removed below the service alignment.

In areas where the exposed subgrade in the service excavation is disturbed below the subgrade level, the disturbed material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular A or Granular B Type II. The use of clear crushed stone as a sub-bedding material should not be permitted.

The granular sub-bedding materials should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. To provide adequate support for the services in the long term in areas where subexcavation of material is required below design subgrade level, the excavations should be sized to allow a 1 horizontal to 1 vertical spread of granular material down and out from the bottom of the service.

6.10.5 Trench Bedding, Cover and Compaction

The bedding and cover for the proposed services should consist of at least 150 and 300 millimetres of OPSS Granular A, respectively, and be placed in accordance with the applicable Ontario Standard Drawings (OPSD) for the type of underground service installed. The use of 19 millimetre clear stone is not recommended as bedding or cover.

The granular bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value of the selected material.

6.10.6 Trench Backfill

In areas where the service trench will be located below or in close proximity to existing or future areas of hard surfacing (i.e., access roadways and parking), acceptable native and existing fill materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent hard surfaced area. The design depth of frost penetration in exposed areas is indicated to be 1.8 metres below finished grade. Where native or existing fill backfill is used, it should match the materials exposed on the trench walls. Backfill below the zone of seasonal frost penetration could consist of either acceptable native material or imported granular material conforming to OPSS Granular B Type I.

It is anticipated that most of the inorganic overburden materials encountered during the subsurface investigation will be acceptable for reuse as trench backfill. Topsoil or other organic material should be wasted.

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, curbs, and parking areas, the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The specified density for compaction of the backfill materials may be reduced where the trench backfill is not located below or in close proximity to existing or future areas of hard surfacing and/or structures, provided that some settlement above the trench is acceptable.

Alternatively, consideration could be given to implementing one or a combination of the following measures to reduce post construction settlement above the trenches, depending on the weather conditions encountered during the construction:

- Allow the overburden materials to dry prior to placement and compaction; and/or,
- Reuse any wet materials in the lower part of the trenches and make provision to defer final paving of surface course (i.e., the Superpave 12.5 asphaltic concrete) in the roadway for 3 months, or longer, to allow the trench backfill settlement to occur and thereby improve the final paved area appearance.

6.10.7 Seepage Barriers

The granular bedding in the service trench could act as a “French Drain”, which could promote groundwater lowering. Seepage barriers should therefore be installed along the service trenches at strategic locations at a horizontal spacing of no more than 100 metres. The seepage barriers should begin at subgrade level and extend vertically through the granular pipe bedding and granular surround to within the native backfill materials, and horizontally across the full width of the service trench excavation. The seepage barriers could consist of 1.5 metre wide dykes of compacted weathered silty clay. The weathered silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Recommendations for the locations of the seepage barriers could be provided as the design progresses and further details are provided.

6.10.8 Thrust Restraint for Watermain and Force Mains

Based on the results of the boreholes, the subsurface conditions at the depths of the proposed watermains and force mains may consist of silty sand/sandy silt. In areas where the subgrade below the thrust block is disturbed, the disturbed/unsuitable material should be removed and replaced with a layer of compacted granular material (i.e., engineered fill), such as that meeting

OPSS Granular B Type II. Engineered fill for thrust block restraint should extend at least 1.5 metres horizontally beyond the thrust block and should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The following parameters could be used for design purposes:

Coefficient of friction between granular backfill and smooth PVC pipe:	0.25
Allowable bearing pressure for thrust blocks bearing on native soil or on a pad of compacted granular material on native soil:	50 kilopascals

The above allowable bearing pressure for the thrust blocks assumes that they are vertical and bear on native, undisturbed soil or compacted engineered fill. The bearing pressure should be reduced if the soil is not excavated vertically, or if the soil is disturbed.

6.10.9 Post-Construction Settlement

The design of the services should consider some differential settlement between the various areas of the site. The amount of differential settlement is expected to be less than 25 millimetres.

6.11 Paved Laneways/Parking Areas

6.11.1 Subgrade Preparation

In preparation for construction of the proposed paved laneway and parking areas at the site, and for predictable performance of the pavement, all fill material and any soft, wet or deleterious materials should be removed from these areas. Prior to placing any grade raise and/or granular material within these areas, the exposed subgrade should be heavily proof rolled and inspected and approved by geotechnical personnel. Any soft areas evident from the proof rolling should be subexcavated and replaced with suitable, compacted earth borrow approved by the geotechnical engineer. Similarly, should it be necessary to raise the grades in these areas, material which meets OPSS specifications for Select Subgrade Material or Earth Borrow could be used. The Select Subgrade Material or Earth Borrow should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment.

Truck traffic should be avoided on the native soil subgrade or the trench backfill within the parking space area, especially under wet conditions.

6.11.2 Pavement Structure

For the parking/laneway areas to be used by light vehicles (cars, etc.), the following minimum pavement structure is recommended:

- 50 millimetres of hot mix asphaltic concrete (50 millimetre lift of Superpave 12.5); over,

- 150 millimetres of OPSS Granular A base; over,
- 300 millimetres of OPSS Granular B; Type II subbase.

For parking/laneway areas to be used by heavy truck traffic, the recommended minimum pavement structure is:

- 100 millimetres of hot mix asphaltic concrete (40 millimetres of Superpave 12.5 over 60 millimetres of Superpave 19.0); over,
- 150 millimetres of OPSS Granular A base; over,
- 450 millimetres of OPSS Granular B, Type II subbase.

The above pavement structures assume that the access roadway and parking lot subgrade surfaces are prepared as described in this report. If the subgrade surfaces become disturbed or wetted due to construction operations or precipitation, the granular subbase thicknesses given above may not be adequate and it may be necessary to increase the thickness of the subbase and/or to incorporate a woven geotextile separator between the subgrade surfaces and the granular subbase material. The adequacy of the design pavement thicknesses should be assessed by geotechnical personnel at the time of construction.

6.11.3 Granular Material Placement

The pavement granular materials should be compacted in maximum 300 millimetre thick lifts to at least 99 percent of standard Proctor maximum dry density using suitable vibratory compaction equipment.

6.11.4 Asphalt Cement Type

Performance grade PG 58-34 asphalt cement should be specified for Superpave asphaltic concrete mixes.

6.11.5 Transition Treatments

In areas where the new pavement structure will abut existing pavements, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

6.11.6 Pavement Drainage

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. The subgrade surfaces should be crowned and shaped to drain to the ditches and/or catch basins to promote drainage of the pavement granular materials.

6.12 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in samples of the native soil recovered from boreholes 23-02 and 23-03 was 72 and 123 micrograms per gram. According to the Canadian Standards Association "Concrete Materials and Methods of Concrete Construction" (CSA A23.1-14 Table 3), the degree of sulphate exposure stemming from the sandy silt can be considered as low. Therefore, any concrete in contact with the native soils at this site should be batched with appropriate cementing materials. Other factors (structurally reinforced or non-structurally reinforced, freeze-thaw environment, chloride exposure, etc.) should also be considered in selecting the cementing materials as well as the associated air entrainment and concrete mix proportions for any concrete.

Based on the pH and resistivity of the soil samples, the materials can be classified as non-aggressive towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the native soils should be consulted to ensure that the durability of the intended product is appropriate. It is noted that the corrosivity of the soil could vary throughout the year due to the application of de-icing chemicals. As well, the samples of soils that were analyzed may not be representative of the entire volume of fill and native soils in the construction zone given the apparent variability of these materials.

6.13 Sensitive Marine Clay – Effects of Trees

The site is underlain by silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations, or hard surfaced areas. Therefore, deciduous tree planting may be carried out in accordance with the guidelines identified in the City of Ottawa document titled: "Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines". It is noted that these guidelines are intended for residential subdivisions, not commercial developments, but may be used to guide tree planting at such sites.

The City of Ottawa Tree Planting Guidelines indicates that sensitive marine clay soils with a modified plasticity index of less than 40 percent are considered to have a low/medium potential for soil volume change. Clay soils with a modified plasticity index that exceeds 40 percent are considered to have a high potential for soil volume change.

The modified plasticity index of the sample of weathered crust provided in Table 5.3 is about 28 percent. As such, the potential for soil volume change, as defined by the City of Ottawa, is low/medium.

In accordance with the City of Ottawa Tree Planting Guidelines, tree planting restrictions apply where clay soils with low/medium potential for volume change are present between the underside of footing and a depth of 3.5 metres below the underside of foundations (refer to the City of Ottawa document titled: “Tree Planting in Sensitive Marine Soils - 2017 Guidelines”) – as is likely the case at this site.

According to the City of Ottawa 2017 Tree Planting Guidelines, the tree to foundation setbacks within this development can be reduced to 4.5 metres for small to medium sized trees (i.e., trees with a mature height of less than 14 metres) with further information and recommendations on planting trees near foundations provided in the City of Ottawa Tree Planting in Sensitive Marine Clay Soils – 2017 Guidelines.

7.0 ADDITIONAL CONSIDERATIONS

7.1 Effects of Construction Induced Vibration

Some of the construction operations (such as excavation, granular material compaction, etc.) will cause ground vibration on and off the site. The vibrations will attenuate with distance from the source but may be felt at nearby structures. Assuming that any excavating is carried out in accordance with the guidelines in this report, the magnitude of the vibrations will be much less than that required to cause damage to the nearby structures or services in good condition but may be felt at the nearby structures.

7.2 Existing Underground Services

The structures could be installed in or near areas where existing underground utilities may be present (this has not been assessed by GEMTEC).

To reduce the risk of impact to existing underground services, it is recommended that the excavations do not encroach within a line extending downwards and outwards at an inclination of 1 vertical to 2 horizontal from the base of the existing services (or edge of structural foundations). Where this is not possible, a detailed assessment by the geotechnical engineer is recommended.

It is recommended that an assessment be carried out in regards to the position of any existing services that are sensitive to movement within the above limits, and the tolerable limits of movement for the services be established in advance of construction. The contractor should be required to clearly identify the proposed method of maintaining movements below the tolerable limits, and the actions to be carried out should movements approach the limits.

7.3 Winter Construction

The native soils that exist at this site are considered to be frost susceptible and prone to ice lensing. In the event that construction is required during freezing temperatures, the native soils within excavations should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

The excavations should be opened for as short a time and limited in area as is reasonably practicable.

8.0 CLOSURE

We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Matthew Rainville, C.E.T.
Senior Technologist

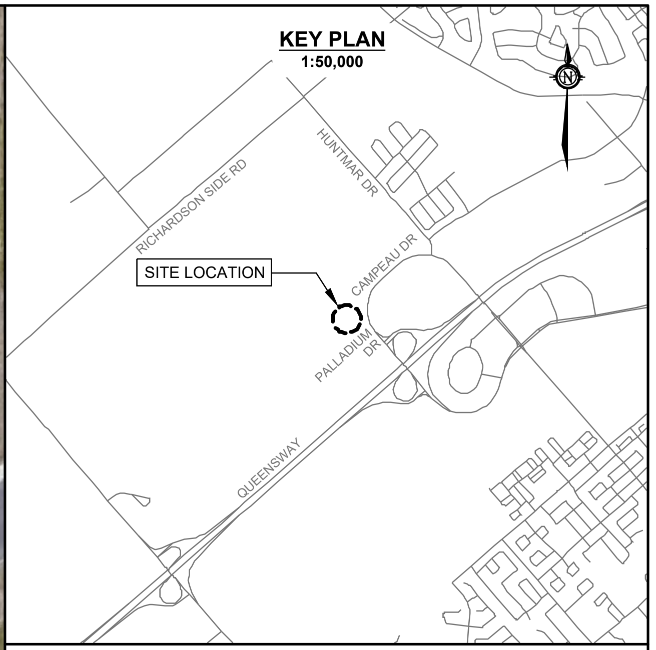
William (Bill) Cavers., P.Eng.
Principal Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
W. CAVERS
100077934
June 20, 2023
PROVINCE OF ONTARIO

SE/AP/MR/BC

KEY PLAN

1:50,000



LEGEND

BH/TP # ← BH/TP ID

XX.XX ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

⊕	**BOREHOLE LOCATION IN PLAN** (current investigation by GEMTEC)
⊕	**APPROXIMATE PREVIOUS BOREHOLE LOCATION** (previous investigation by Patterson Group, November 2014)
⊕	**APPROXIMATE PREVIOUS BOREHOLE LOCATION** (previous investigation by Patterson Group, December 2010)

GENERAL NOTE(S)

1. Contains information licensed under the Open Government Licence – Ontario.
2. Maps Data: Google, @2023 CNES / Airbus, First Base Solutions, Maxar Technologies.
3. Geographic dataset source: Ontario GeoHub.
4. "Site Plan" provided by Allan Stone Architect, May 17, 2023.

SCALE

1:1000

0 20 40 60m

DRAWING

BOREHOLE LOCATION PLAN

CLIENT

3095 PALLADIUM GP INC.

PROJECT

3095 PALLADIUM DRIVE
OTTAWA, ONTARIO

DRAWN BY C.Z.	CHECKED BY M.R.
PROJECT NO. 102670.002	REVISION NO. 0
DATE MAY 2023	FIGURE NO. FIGURE 1

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3. **Complete Report:** This report is of a summary nature and is not intended to stand alone without reference to the instructions given to GEMTEC by the Client, communications between GEMTEC and the Client and to any other reports prepared by GEMTEC for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. GEMTEC cannot be responsible for use of portions of the report without reference to the entire report.
4. **Basis of Report:** This Report has been prepared for the specific site, development, design objectives and purposes that were described to GEMTEC by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document, subject to the limitations provided herein, are only valid to the extent that this report expressly addresses the proposed development, design objectives and purposes. Any change of site conditions, purpose or development plans may alter the validity of the report and GEMTEC cannot be responsible for use of this report, or portions thereof, unless GEMTEC is requested to review any changes and, if necessary, revise the report.
5. **Time Dependence:** If the proposed project is not undertaken by the Client within 18 months following the issuance of this report, or within the timeframe understood by GEMTEC to be contemplated by the Client, the guidance and recommendations within the report should not be considered valid unless reviewed and amended or validated by GEMTEC in writing.
6. **Use of This Report:** The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without GEMTEC's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, GEMTEC may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process.

   Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.
7. **No Legal Representations:** GEMTEC makes no representations whatsoever concerning the legal significance of its findings, or as to other legal matters touched on in this report, including but not limited to, ownership of any property, or the application of any law to the facts set forth herein. With respect to regulatory compliance issues, regulatory statutes are subject to interpretation and change. Such interpretations and regulatory changes should be reviewed with legal counsel.
8. **Decrease in Property Value:** GEMTEC shall not be responsible for any decrease, real or perceived, of the property or site’s value or failure to complete a transaction, as a consequence of the information contained in this report.
9. **Reliance on Provided Information:** The evaluation and conclusions contained in this report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations. information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in this report as a result of misstatements, omissions,

misrepresentations. or fraudulent acts of the Client or other persons providing information relied on by us. We are entitled to rely on such representations, information and instructions and are not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

10. **Investigation Limitations:** Site investigation programs are a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions but even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions.

    The data derived from the site investigation program and subsequent laboratory testing are interpreted by trained personnel and extrapolated across the site to form an inferred geological representation and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Conditions between and beyond the borehole/test hole locations may differ from those encountered at the borehole/test hole locations and the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. Accordingly, GEMTEC does not warrant or guarantee the exactness of of the subsurface descriptions.

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

    In addition, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

11. **Sample Disposal:** GEMTEC will dispose of all uncontaminated soil and/or rock samples 60 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fill materials or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

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

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

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

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

APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Symbols

RECORD OF BOREHOLE 23-01

CLIENT: 3095 Palladium GP Inc.
PROJECT: 3095 Palladium Drive, Ottawa, ON
JOB#: 102670.002
LOCATION: See Borehole Location Plan, Figure 1

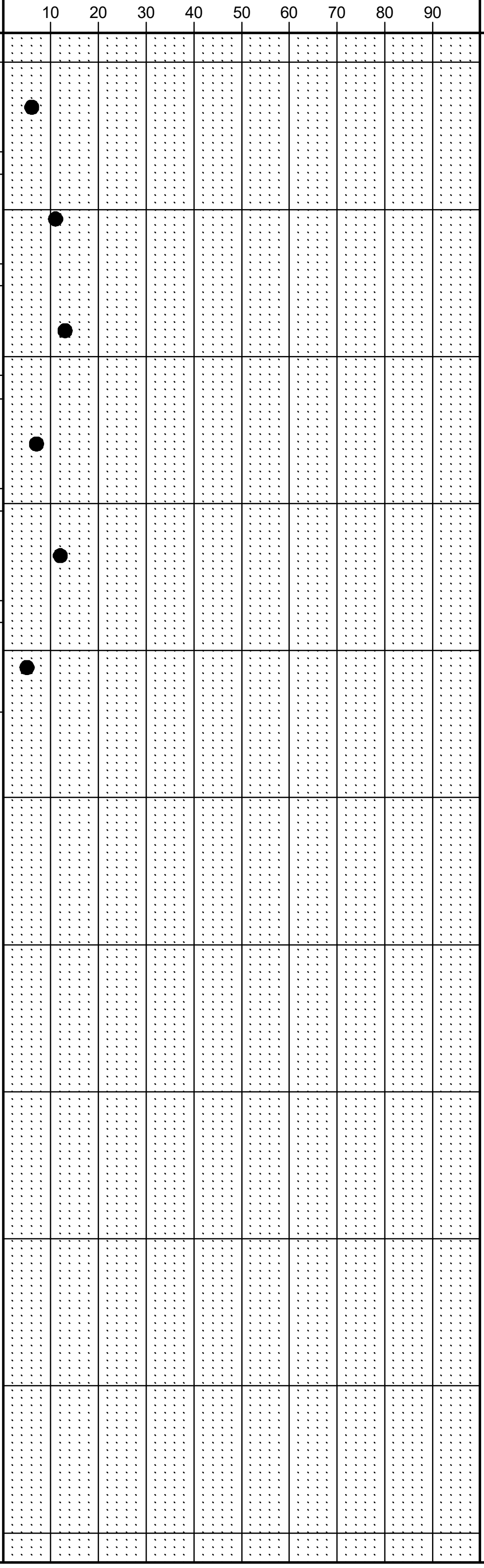
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: May 23 2023

BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	103.84
Dark brown silty sand and sandy silt, with organic material (TOPSOIL / FILL MATERIAL)	103.61 0.23
Grey sandy silty clay, some gravel cobbles and boulders (FILL MATERIAL)	102.72 1.12
Very stiff, grey brown silty clay, trace sand (WEATHERED CRUST)	102.06 1.78
Compact to loose, grey SILTY SAND to SANDY SILT	99.40 4.44
Practical auger refusal End of borehole	

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	381	6
2	SS	25	11
3	SS	508	13
4	SS	406	7
5	SS	483	12
6	SS	406	5

● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA ┼ NATURAL ⊕ REMOULDED
WATER CONTENT, % W_P ⊢ W ⊣ W_L



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

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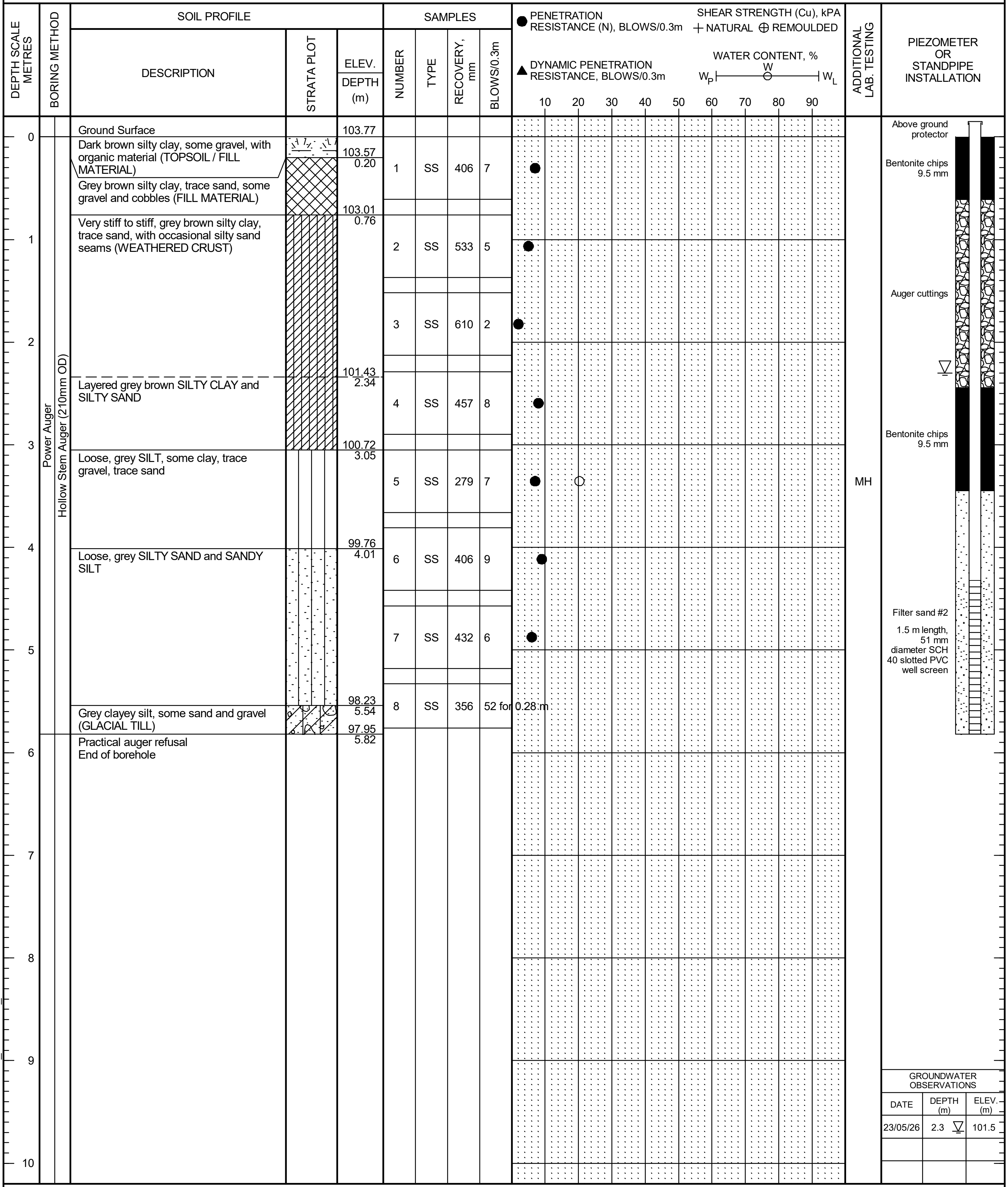
LOGGED: A.N.
CHECKED: M.R.

RECORD OF BOREHOLE 23-02

CLIENT: 3095 Palladium GP Inc.
PROJECT: 3095 Palladium Drive, Ottawa, ON
JOB#: 102670.002
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: May 23 2023



SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	103.77					
Dark brown silty clay, some gravel, with organic material (TOPSOIL / FILL MATERIAL)	103.57 0.20	1	SS	406	7	
Grey brown silty clay, trace sand, some gravel and cobbles (FILL MATERIAL)						
Very stiff to stiff, grey brown silty clay, trace sand, with occasional silty sand seams (WEATHERED CRUST)	103.01 0.76	2	SS	533	5	
		3	SS	610	2	
Layered grey brown SILTY CLAY and SILTY SAND	101.43 2.34	4	SS	457	8	
Loose, grey SILT, some clay, trace gravel, trace sand	100.72 3.05	5	SS	279	7	MH
Loose, grey SILTY SAND and SANDY SILT	99.76 4.01	6	SS	406	9	
		7	SS	432	6	
Grey clayey silt, some sand and gravel (GLACIAL TILL)	98.23 5.54	8	SS	356	52 for 0.28 m	
Practical auger refusal End of borehole	97.95 5.82					

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/05/26	2.3 ∇	101.5

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Consulting Engineers and Scientists

LOGGED: A.N.

CHECKED: M.R.

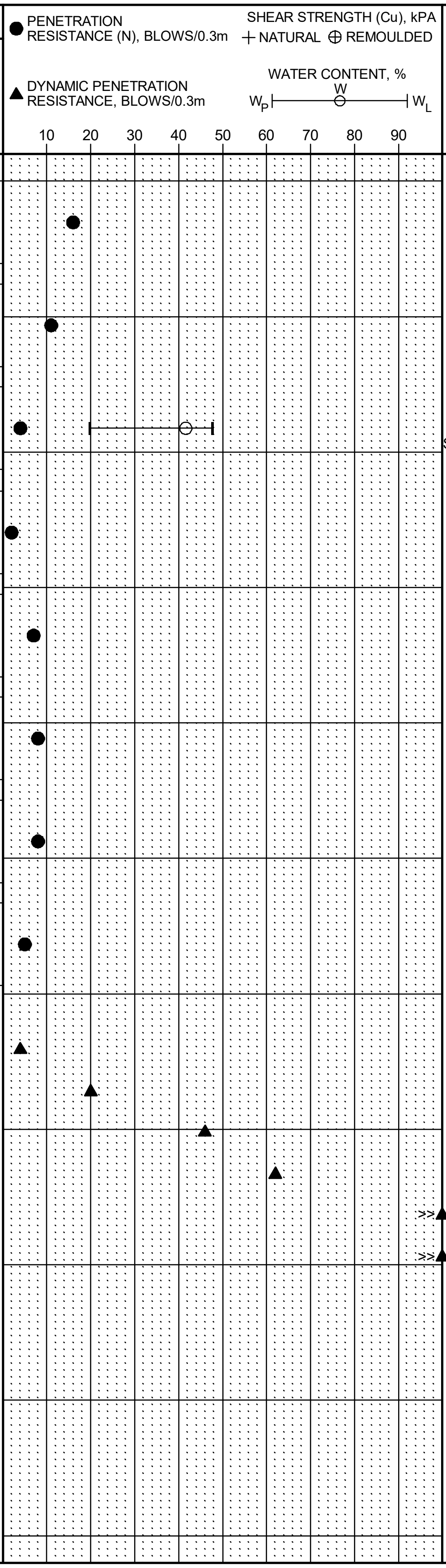
RECORD OF BOREHOLE 23-03

CLIENT: 3095 Palladium GP Inc.
PROJECT: 3095 Palladium Drive, Ottawa, ON
JOB#: 102670.002
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: May 23 2023

BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)
	Ground Surface	103.63
Power Auger – Hollow Stem Auger (210mm OD)	Compact, grey silty sand, some gravel, with cobbles (FILL MATERIAL)	
	Very stiff to stiff, grey brown silty clay trace sand (WEATHERED CRUST)	102.79 0.84
	Layered, grey brown SILTY SAND and SILTY CLAY	100.43 3.20
	Grey SILT, some clay, trace to some gravel	99.64 3.99
	Grey SILTY SAND and SANDY SILT	98.93 4.70
DCPT	Dynamic Cone Penetration Test (DCPT) conducted	97.53 6.10
	Refusal to DCPT advancement End of borehole	95.51 8.12

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	483	16	
2	SS	381	11	
3	SS	610	4	MH, SG, SL
4	SS	610	2	
5	SS	610	7	
6	SS	432	8	
7	SS	483	8	
8	SS	508	5	



PIEZOMETER OR STANDPIPE INSTALLATION

Borehole backfilled with auger cuttings

LOGGED: A.N.

CHECKED: M.R.

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

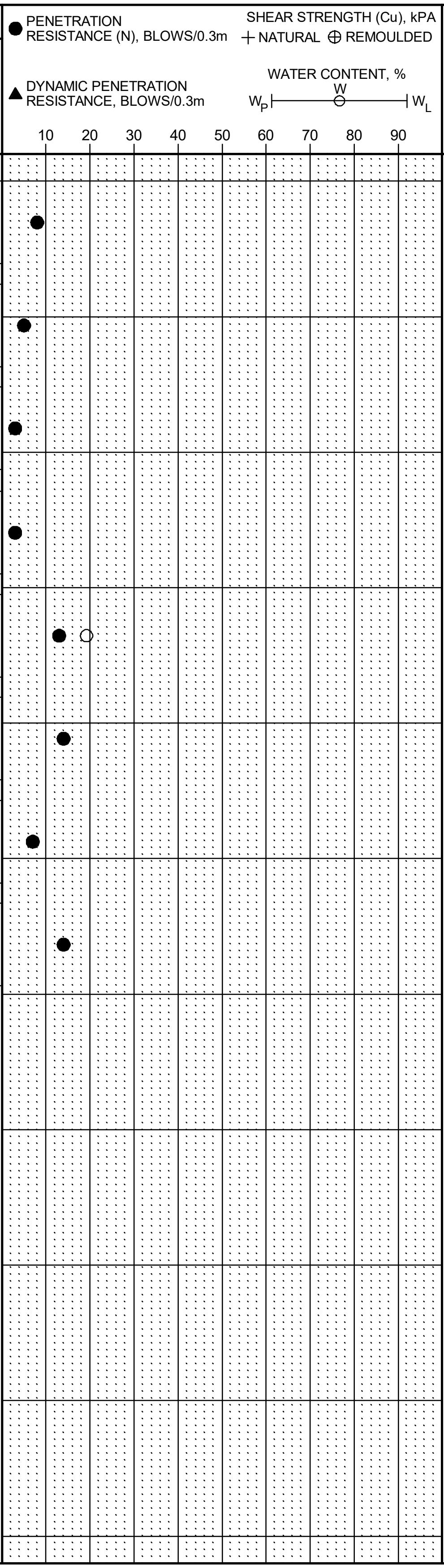
RECORD OF BOREHOLE 23-04

CLIENT: 3095 Palladium GP Inc.
PROJECT: 3095 Palladium Drive, Ottawa, ON
JOB#: 102670.002
LOCATION: See Borehole Location Plan, Figure 1

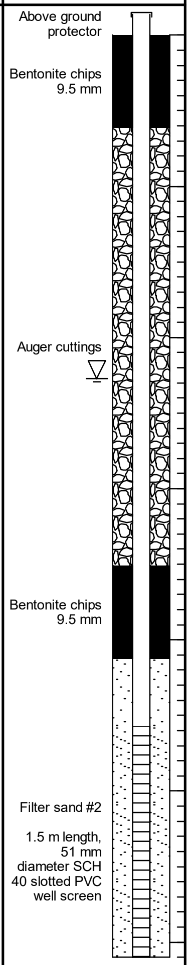
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: May 23 2023

BORING METHOD: Power Auger — Hollow Stem Auger (210mm OD)

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
Ground Surface	103.84					
Loose, brown silty sand, trace gravel (FILL MATERIAL)		1	SS	533	8	
Possible former topsoil	102.93 0.91 102.82	2	SS	508	5	
Very stiff to stiff, grey brown silty clay trace sand, with occasional silty sand seams (WEATHERED CRUST)		3	SS	457	3	
		4	SS	610	3	
Compact, grey SILT, some clay, trace gravel, trace sand to SANDY SILT	100.64 3.20	5	SS	457	13	MH
		6	SS	457	14	
Compact, grey SILTY SAND, with sandy silt seams	99.06 4.78	7	SS	406	7	
		8	SS	431	14	
Grey clayey sandy silt, some gravel (GLACIAL TILL)	98.00 5.84					
End of borehole	97.74 6.10					



PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
23/05/26	2.3 ∇	101.6

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Consulting Engineers and Scientists

LOGGED: A.N.
CHECKED: M.R.

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

SAMPLE TYPES	
AS	Auger sample
CA	Casing sample
CS	Chunk sample
BS	Borros piston sample
GS	Grab sample
MS	Manual sample
RC	Rock core
SS	Split spoon sampler
ST	Slotted tube
TO	Thin-walled open shelby tube
TP	Thin-walled piston shelby tube
WS	Wash sample

SOIL TESTS	
w	Water content
PL, w_p	Plastic limit
LL, w_L	Liquid limit
C	Consolidation (oedometer) test
D_R	Relative density
DS	Direct shear test
G_S	Specific gravity
M	Sieve analysis for particle size
MH	Combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	Organic content test
UC	Unconfined compression test
γ	Unit weight

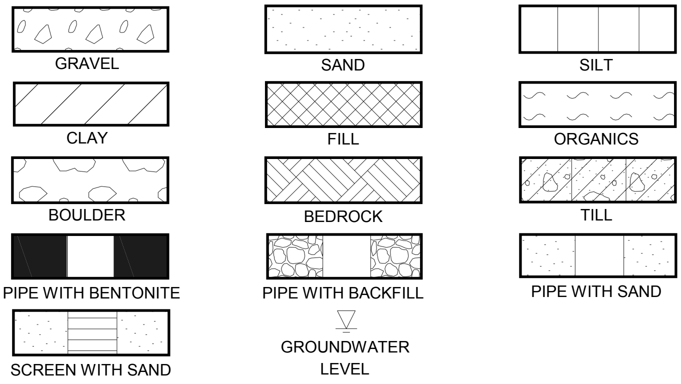
PENETRATION RESISTANCE

Standard Penetration Resistance, N
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 millimetres (30 in.) required to drive a 50 mm split spoon sampler for a distance of 300 mm (12 in.). For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

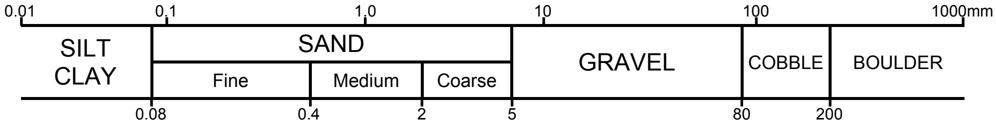
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to ‘A’ size drill rods for a distance of 300 mm (12 in.).

WH	Sampler advanced by static weight of hammer and drill rods
WR	Sampler advanced by static weight of drill rods
PH	Sampler advanced by hydraulic pressure from drill rig
PM	Sampler advanced by manual pressure

COHESIONLESS SOIL Compactness		COHESIVE SOIL Consistency	
SPT N-Values	Description	Cu, kPa	Description
0-4	Very Loose	0-12	Very Soft
4-10	Loose	12-25	Soft
10-30	Compact	25-50	Firm
30-50	Dense	50-100	Stiff
>50	Very Dense	100-200	Very Stiff
		>200	Hard



GRAIN SIZE



DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

0	10	20	35
TRACE	SOME	ADJECTIVE	noun > 35% and main fraction
trace clay, etc	some gravel, etc.	silty, etc.	sand and gravel, etc.

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APPENDIX B

Record of Borehole and Test Pit Sheets by Others

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Retail Centre - Block 13
Palladium Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE November 12, 2014

FILE NO. PG3115

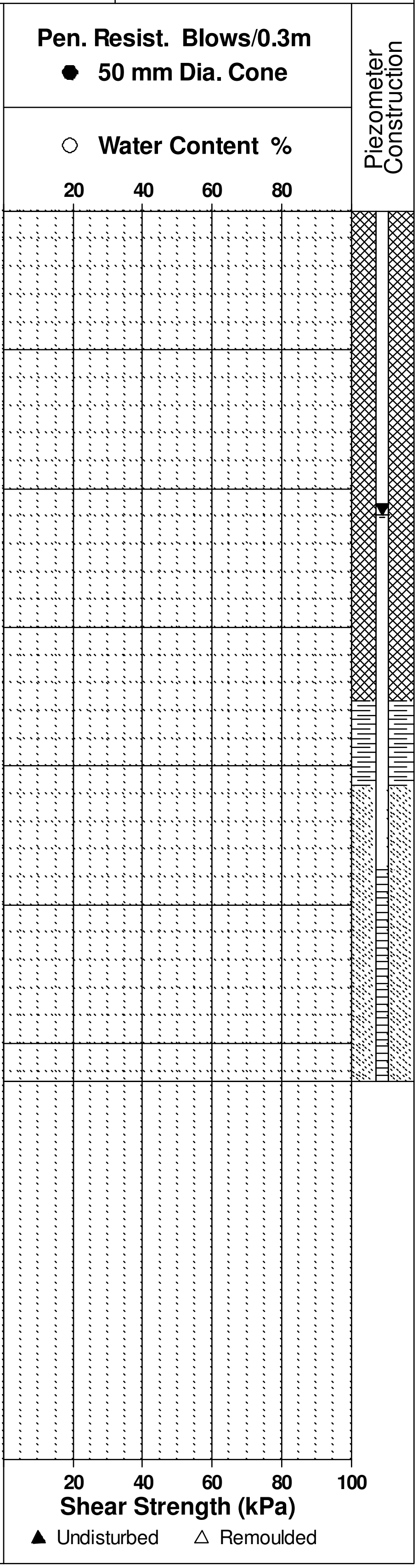
HOLE NO. BH31

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.22
TOPSOIL 0.18							
Stiff, brown **SILTY CLAY,** trace sand 0.66		SS	1	42	5		
Loose to very loose, brown **SILTY SAND** - very loose to loose and grey by 3.0m depth		SS	2	100	7	1	102.22
						2	101.22
		SS	3	100	4		
						3	100.22
		SS	4	100	9		
		SS	5	33	2	4	99.22
		SS	6	54	9	5	98.22
		SS	7	71	9		
6.00						6	97.22
GLACIAL TILL: Grey silty sand with gravel and cobbles 6.27		SS	8	57	50+		

End of Borehole

Practical refusal to augering at 6.27m depth

(GWL @ 2.19m-Nov. 18, 2014)



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Kanata West Retail Centre - Block 13
Palladium Drive, Ottawa, Ontario

DATUM Ground surface elevations provided by Stantec Geomatics Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

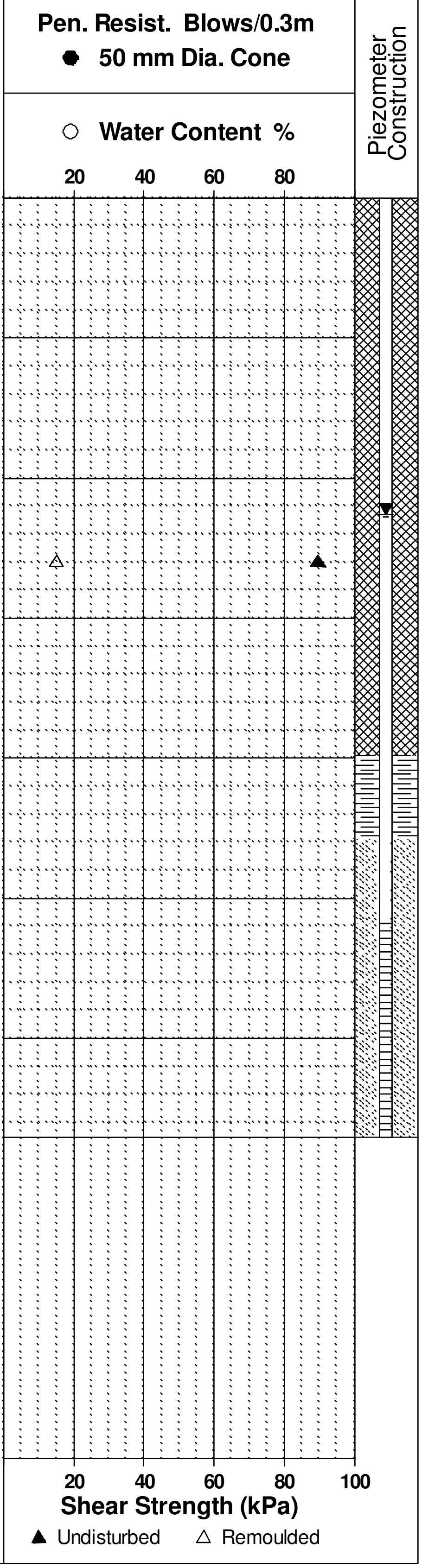
DATE November 12, 2014

FILE NO. **PG3115**

HOLE NO. **BH32**

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
TOPSOIL 0.18				
Very stiff, brown **SILTY CLAY**	SS	1	42	10
	SS	2	100	8
	SS	3	100	6
2.97				
Loose, grey **SILTY SAND**	SS	4	46	5
	SS	5	50	7
	SS	6	54	5
5.33				
GLACIAL TILL: Grey silty sand with gravel and cobbles	SS	7	100	9
	SS	8	8	2
6.70				
End of Borehole (GWL @ 2.26m-Nov. 18, 2014)				

DEPTH (m)	ELEV. (m)
0	103.54
1	102.54
2	101.54
3	100.54
4	99.54
5	98.54
6	97.54



patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Kanata West Business Park - Huntmar Road
Ottawa, Ontario

DATUM Ground suface elevations provided by Stantec Geomatics

REMARKS

BORINGS BY CME 55 Power Auger

DATE December 1, 2010

FILE NO. PG0912

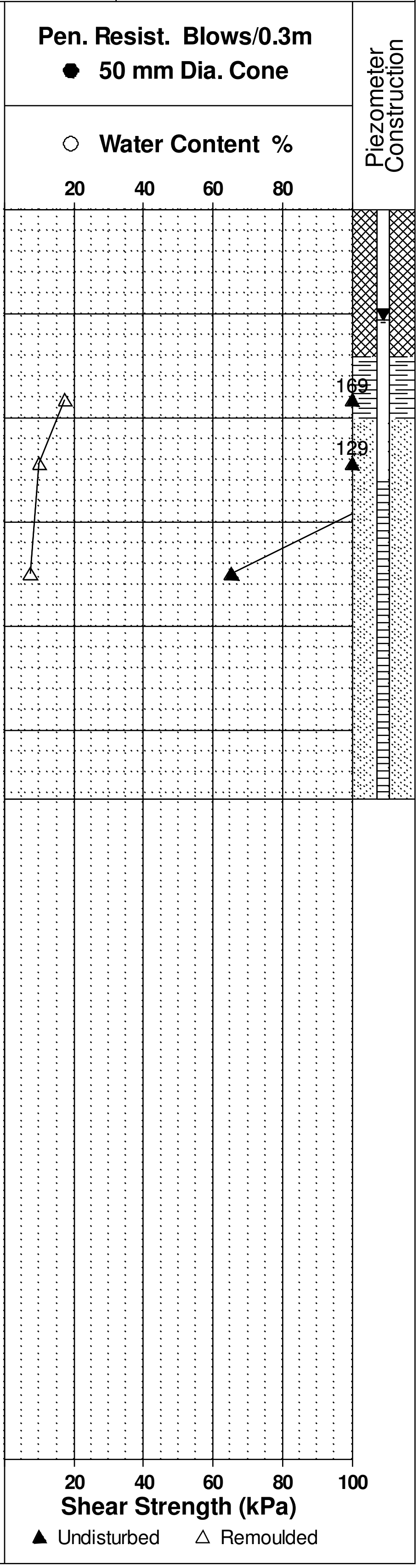
HOLE NO. BH 4-10

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	103.69
TOPSOIL 0.25							
Very stiff, brown **SILTY CLAY**		SS	1	83	3	1	102.69
						2	101.69
2.59							
Loose to compact, grey **SANDY SILT**						3	100.69
		SS	2	50	6	4	99.69
4.82							
GLACIAL TILL: Very dense, grey silty sand with gravel, cobbles and boulders		SS	3	33	13	5	98.69
5.66		SS	4	67	50+		

End of Borehole

Practical refusal to augering @ 5.66m depth

(GWL @ 1.06m-Dec. 22/10)



APPENDIX C

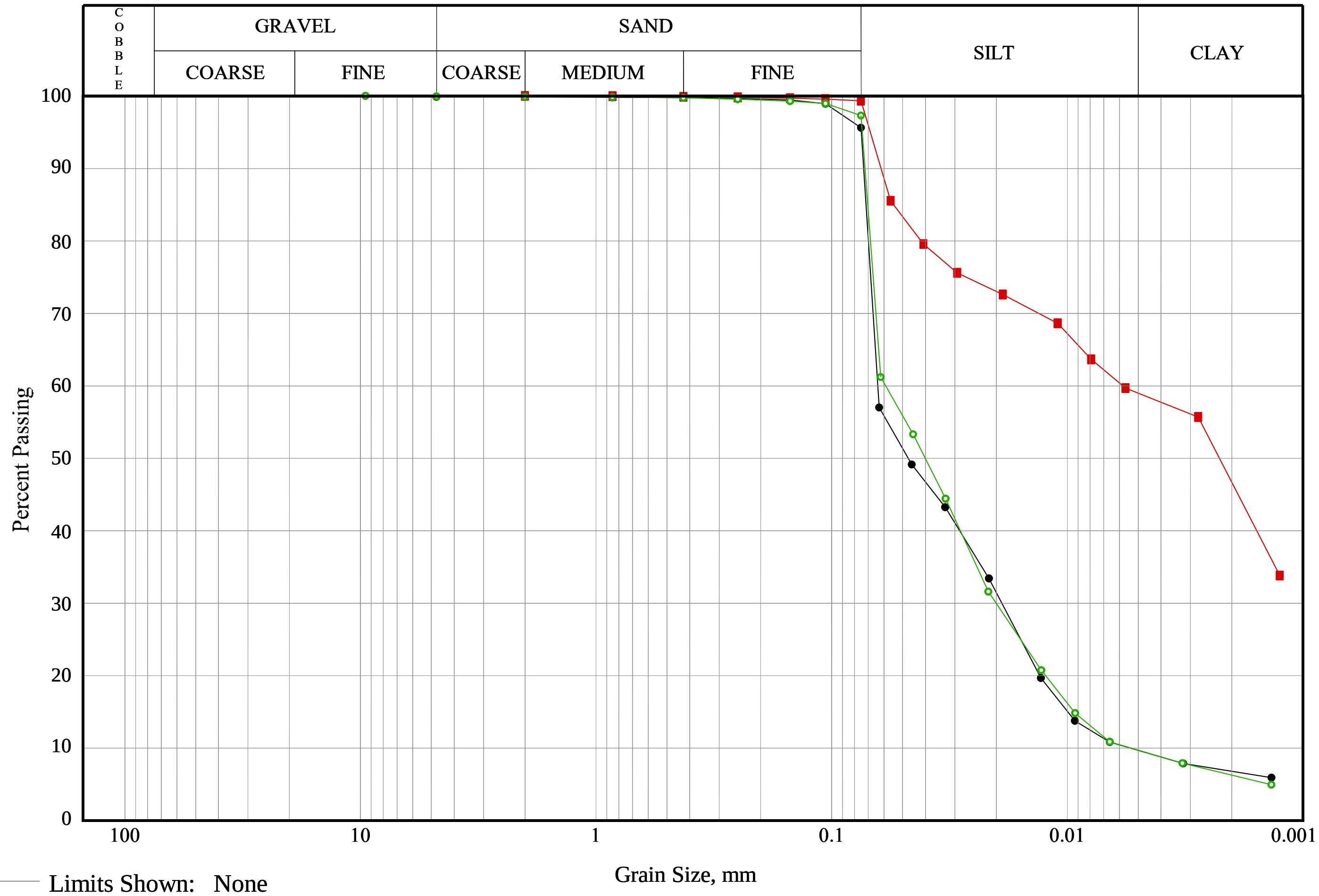
Laboratory Test Results

Report to: 3095 Palladium GP Inc. c/o Quaestus
Project: 102670.002 (June 20, 2023)

GEMTEC
Consulting Engineers and Scientists

Client: 3095 Palladium GP Inc.
Project: 3095 Palladium Drive, Ottawa, ON
Project #: 102670.002

Soils Grading Chart (LS-702/ ASTM D-422)



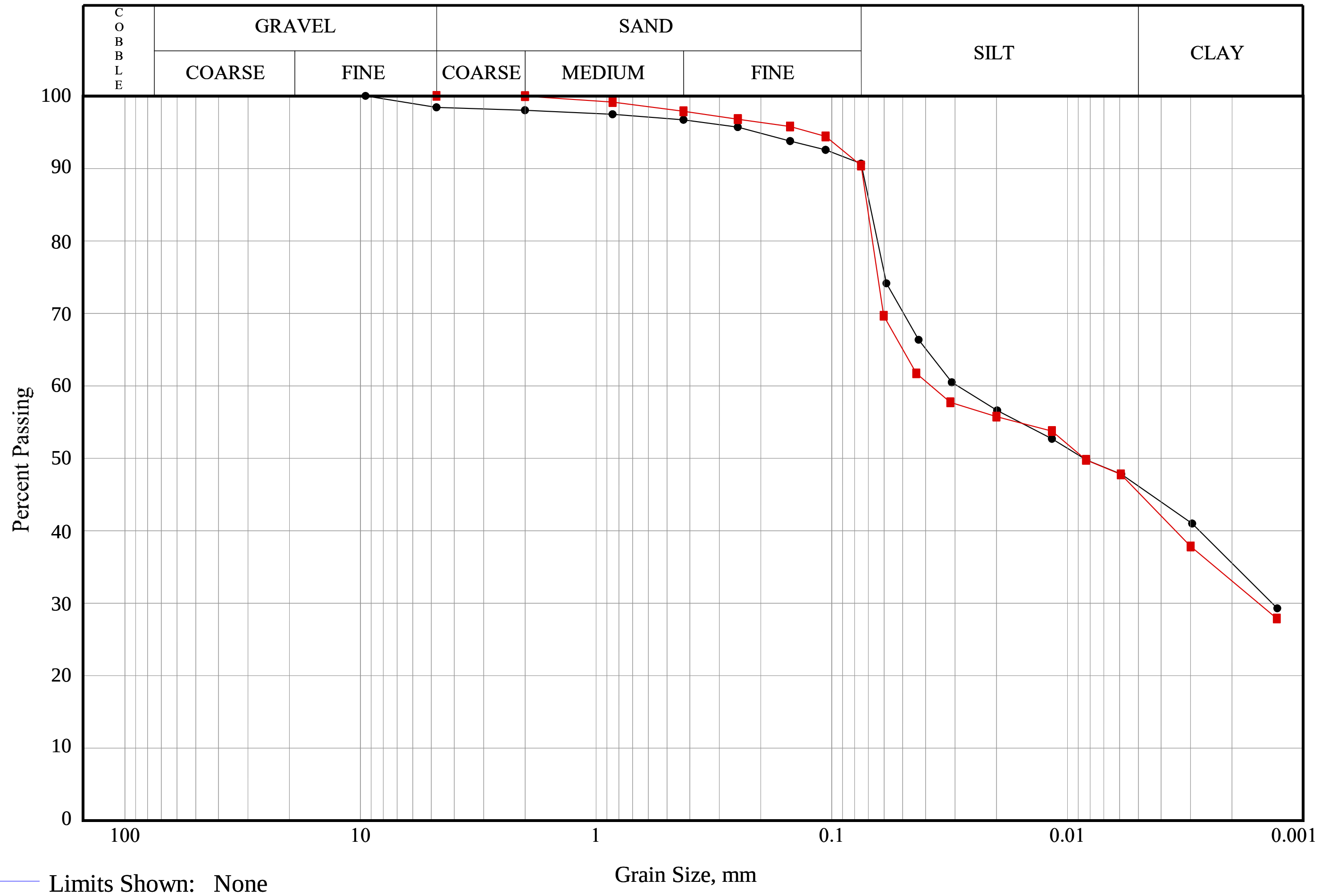
Limits Shown: None

Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Native Silt, some clay, trace gravel/sand	23-02	SA 5	3.1-3.7	0.1	4.3	85.9	9.7
—■—	Native Silty Clay	23-03	SA 3	1.5-2.1	0.0	0.7	40.3	59.0
—○—	Native Silt, some clay, trace gravel/sand	23-04	SA 5	3.1-3.7	0.1	2.6	87.6	9.7

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
—●—	Silt , trace gravel, trace sand, trace clay	N/A	0.01	0.01	0.02	0.05	0.06	0.07	85.9
—■—	Clay and silt , trace sand	CL	---	---	---	0.00	0.01	0.05	40.3
—○—	Silt , trace gravel, trace sand, trace clay	N/A	0.01	0.01	0.02	0.04	0.06	0.07	87.6

GEMTEC Consulting Engineers and Scientists	Client: 3095 Palladium GP Inc. Project: 3095 Palladium Drive, Ottawa, ON Project #: 102670.002	Soils Grading Chart (LS-702/ ASTM D-422)

Soils Grading Chart (LS-702/ ASTM D-422)



Limits Shown: None

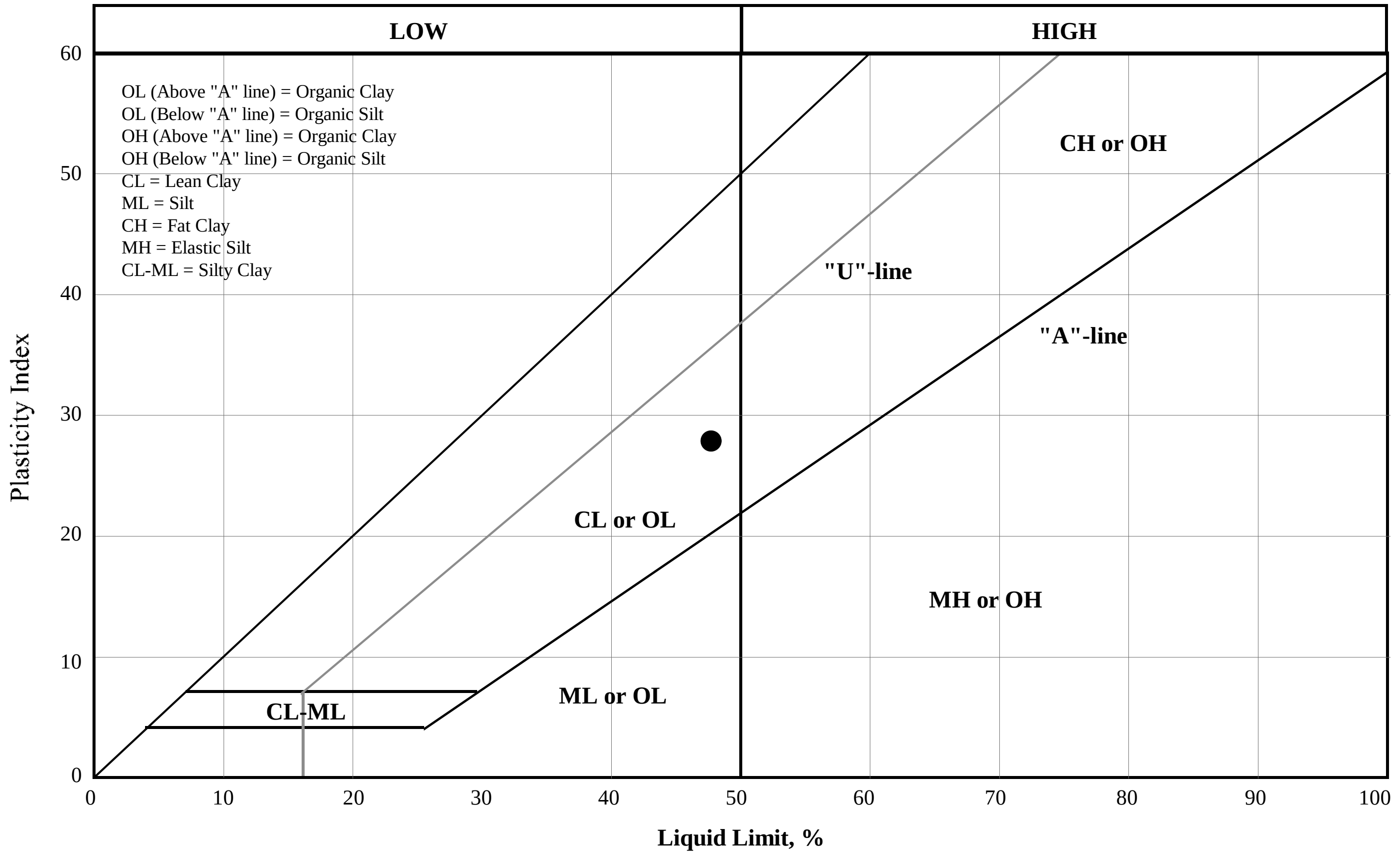
Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	Native Silty Clay	GP23-2	1	0.8-0.9	1.6	7.8	44.5	46.2
■	Native Silty Clay	GP23-4	1	1.0-1.10	0.0	9.6	45.1	45.3

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●	Clay and silt , trace gravel, trace sand	N/A	---	---	0.00	0.01	0.03	0.07	44.5
■	Clay and silt , trace sand	N/A	---	---	0.00	0.01	0.04	0.07	45.1

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

Client: 3095 Palladium GP Inc.
Project: 3095 Palladium Drive, Ottawa, ON
Project #: 102670.002

Plasticity Chart (D4318)



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	23-03	SA 3	1.5-2.1	47.7	19.8	27.9	☐	41.62

Note: More information available upon request

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www.gemtec.ca

Shrinkage Limit

ASTM D4943

Volume of Shrinkage Dish

Mass of Glass Plate (g):	20.82	20.84
Mass of Shrinkage Dish (g) (m):	20.84	20.92
Mass of Shrinkage Dish and Grease(g) (m):	37.35	37.35
Mass of Shrinkage Dish, Plate, Grease and Water (g):	75.43	75.52
Mass of Water (g):	17.26	17.33
Volume of Shrinkage Dish:	17.00	17.00

Test Specimen

Specimen Dish:	SL8	SL2
Mass of Shrinakge Dish, m (g):	20.82	20.84
Mass of Shrinkage Dish and Grease, m_{dxg} (g):	20.84	20.92
Mass of Shrinkage Dish and Wet Soil, m_w (g):	50.13	50.17
Mass of Shrinkage Dish and Dry Soil, m_d (g):	40.53	40.51
Mass of Wax-Coated Soil in Air, m_{sxa} (g):	20.16	20.21
Mass of Wax-Coated Soil in Water, m_{sxw} (g):	9.4	9.3

Calculated Shrinkage Limit

Specimen Dish:	SL8	SL2
Mass of Dry Soil, m_s (g):	19.71	19.67
Water Content of Soil when Placed in Dish, w (%):	48.71	49.11
Mass of Water Displaced by Wax-Coated Soil, m_{wsx} (g):	10.76	10.91
Volume of Dry Soil and Wax, V_{dx} (cm^3):	10.76	10.91
Mass of Wax, m_x (g):	0.45	0.54
Volume of Wax, V_x (cm^3):	0.50	0.60
Volume of Dry Soil, V_d (cm^3):	10.26	10.31
Shrinkage Limit, SL:	13.19	13.78
Average Shrinkage Limit, $Sl_{avg.}$:	13.48	

Specific Gravity of Wax = 0.908 at15.5°C

Specific Gravity of Wax = 0.900 at 20°C

Density of Water (g/cm^3) = 1.000 (g/cm^3)

Project No.: 102670.002	**Tested By: J. Kaur**
Project Name: 3095 Palladium Dr., Ottawa, ON	**Checked By: K.Smith**
Date Tested: June 5, 2023	**Sample No:BH 23-03 SA3**
Sample Date:	**Source:**
Remarks:	**Depth: 1.5-2.1**

APPENDIX D

Chemical Analysis of Soil Sample Relating to Corrosion

Report to: 3095 Palladium GP Inc. c/o Quaestus
Project: 102670.002 (June 20, 2023)

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Certificate of Analysis

GEMTEC Consulting Engineers and Scientists Limited

32 Steacie Drive
Kanata, ON K2K 2A9
Attn: Matt Rainville

Client PO:
Project: 102670.002
Custody:

Report Date: 2-Jun-2023
Order Date: 26-May-2023

Order #: 2322161

This Certificate of Analysis contains analytical data applicable to the following samples as submitted :

Paracel ID	**Client ID**
2322161-01	BH-23-03 SA-5 3.1-3.7
2322161-02	BH-23-02 SA-3 1.5-2.1

Approved By:

Dale Robertson, BSc
Laboratory Director

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	31-May-23	31-May-23
Conductivity	MOE E3138 - probe @25 °C, water ext	1-Jun-23	1-Jun-23
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	30-May-23	31-May-23
Resistivity	EPA 120.1 - probe, water extraction	1-Jun-23	1-Jun-23
Solids, %	CWS Tier 1 - Gravimetric	31-May-23	1-Jun-23

Certificate of Analysis

Client: **GEMTEC Consulting Engineers and Scientists Limited**

Client PO:

Report Date: 02-Jun-2023

Order Date: 26-May-2023

Project Description: 102670.002

	Client ID:	BH-23-03 SA-5 3.1-3.7	BH-23-02 SA-3 1.5-2.1	-	-
	Sample Date:	23-May-23 14:00	23-May-23 14:30	-	-
	Sample ID:	2322161-01	2322161-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	76.8	67.5	-	-
General Inorganics					
Conductivity	5 uS/cm	209	166	-	-
pH	0.05 pH Units	7.43	7.28	-	-
Resistivity	0.1 Ohm.m	47.8	60.4	-	-
Anions					
Chloride	10 ug/g dry	14	16	-	-
Sulphate	10 ug/g dry	123	72	-	-

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	10	ug/g						
Sulphate	ND	10	ug/g						
General Inorganics									
Conductivity	ND	5	uS/cm						
Resistivity	ND	0.1	Ohm.m						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	10	ug/g	ND			NC	35	
Sulphate	ND	10	ug/g	ND			NC	35	
General Inorganics									
Conductivity	425	5	uS/cm	423			0.5	5	
pH	6.22	0.05	pH Units	6.25			0.5	2.3	
Resistivity	23.5	0.1	Ohm.m	23.7			0.5	20	
Physical Characteristics									
% Solids	84.0	0.1	% by Wt.	84.6			0.8	25	

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	99.0	10	ug/g	ND	99.0	82-118			
Sulphate	101	10	ug/g	ND	101	80-120			

Qualifier Notes:

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable
ND: Not Detected
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.
NC: Not Calculated

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.
Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

APPENDIX E

Infiltration Testing Results

Report to: 3095 Palladium GP Inc. c/o Quaestus
Project: 102670.002 (June 20, 2023)

Guelph Permeameter Calculations for GP23-4

🟩 Input

🟨 Result

Single Head Method (1)

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir): 2.16

Enter water Head Height ("H" in cm): 10

Enter the Borehole Radius ("a" in cm): 3

Enter the soil texture-structure category (enter one of the below numbers): 2

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min): 0.0500

Res Type	2.16
H	10
a	3
H/a	3.333
a*	0.04
C0.01	1.218
C0.04	1.29
C0.12	1.288
C0.36	1.288
C	1.29
R	0.050
Q	0.002
pi	3.142

α*= 0.04 cm^{-1}

C = 1.290234

Q = 0.0018

K_{fs} =	1.04E-06	cm/sec
	6.23E-05	cm/min
	1.04E-08	m/sec
	2.45E-05	inch/min
	4.09E-07	inch/sec

Φ_m = 2.60E-05 cm^2/min

Single Head Method (2)

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir):

Enter water Head Height ("H" in cm):

Enter the Borehole Radius ("a" in cm):

Enter the soil texture-structure category (enter one of the below numbers):

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

Steady State Rate of Water Level Change ("R" in cm/min):

Res Type	0
H	0
a	0
H/a	#DIV/0!
a*	0
C0.01	#DIV/0!
C0.04	#DIV/0!
C0.12	#DIV/0!
C0.36	#DIV/0!
C	0
R	0.000
Q	0
pi	3.1415

α*= 0 cm^{-1}

C = 0

Q = 0

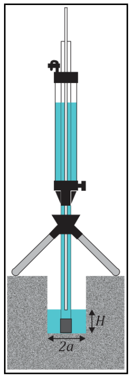
K_{fs} =	#DIV/0!	cm/sec
	#DIV/0!	cm/min
	#DIV/0!	m/ses
	#DIV/0!	inch/min
	#DIV/0!	inch/sec

Φ_m = #DIV/0! cm^2/min

Average

K_{fs} =	#DIV/0!	cm/sec
	#DIV/0!	cm/min
	#DIV/0!	m/s
	#DIV/0!	inch/min
	#DIV/0!	inch/sec

Φ_m = #DIV/0! cm^2/min



Double Head Method

Reservoir Cross-sectional area in cm^2

(enter "35.22" for Combined and "2.16" for Inner reservoir):

Enter the first water Head Height ("H1" in cm):

Enter the second water Head Height ("H2" in cm):

Enter the Borehole Radius ("a" in cm):

Enter the soil texture-structure category (enter one of the below numbers):

1. Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.
2. Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.
3. Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.
4. Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macropors, etc

α*= 0 cm^{-1}

α= #DIV/0!

Steady State Rate of Water Level Change ("R1" in cm/min):

Steady State Rate of Water Level Change ("R2" in cm/min):

Q_1 = 0

Q_2 = 0

C_1 = 0

C_2 = 0

G_1 = #DIV/0!

G_2 = #DIV/0!

G_3 = #DIV/0!

G_4 = #DIV/0!

Res Type:	0
H1/a:	#DIV/0!
H2/a:	#DIV/0!
C1-0.01:	#DIV/0!
C2-0.01:	#DIV/0!
C1-0.04:	#DIV/0!
C2-0.04:	#DIV/0!
C1-0.12:	#DIV/0!
C2-0.12:	#DIV/0!
C1-0.36:	#DIV/0!
C2-0.36:	#DIV/0!
G-Denominator:	0

K_{fs} =	#DIV/0!	cm/sec
	#DIV/0!	cm/min
	#DIV/0!	m/sec
	#DIV/0!	inch/min
	#DIV/0!	inch/sec

Φ_m = #DIV/0! cm^2/min

Θ_{fs} = cm^3/cm^3

Θ_i = cm^3/cm^3

Sorptivity #DIV/0! $(cm\ min^{-½})$

Calculation formulas related to shape factor (*C*). Where H_1 is the first water head height (cm), H_2 is the second water head height (cm), a is borehole radius (cm) and α^* is microscopic capillary length factor which is decided according to the soil texture-structure category. For one-head method, only C_1 needs to be calculated while for two-head method, C_1 and C_2 are calculated (Zang et al., 1998).

Soil Texture-Structure Category	$\alpha^*(cm^{-1})$	Shape Factor
Compacted, Structure-less, clayey or silty materials such as landfill caps and liners, lacustrine or marine sediments, etc.	0.01	$C_1 = \left(\frac{H_2/a}{2.081 + 0.121(H_2/a)}\right)^{0.672}$
Soils which are both fine textured (clayey or silty) and unstructured; may also include some fine sands.	0.04	$C_1 = \left(\frac{H_1/a}{1.992 + 0.091(H_1/a)}\right)^{0.683}$ $C_2 = \left(\frac{H_2/a}{1.992 + 0.091(H_2/a)}\right)^{0.683}$
Most structured soils from clays through loams; also includes unstructured medium and fine sands. The category most frequently applicable for agricultural soils.	0.12	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093(H_1/a)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093(H_2/a)}\right)^{0.754}$
Coarse and gravely sands; may also include some highly structured soils with large and/or numerous cracks, macro pores, etc.	0.36	$C_1 = \left(\frac{H_1/a}{2.074 + 0.093(H_1/a)}\right)^{0.754}$ $C_2 = \left(\frac{H_2/a}{2.074 + 0.093(H_2/a)}\right)^{0.754}$

Calculation formulas related to one-head and two-head methods. Where R is steady-state rate of fall of water in reservoir (cm/s), K_{fs} is Soil saturated hydraulic conductivity (cm/s), Φ_m is Soil matric flux potential (cm^2/s), a^* is Macroscopic capillary length parameter (from Table 2), a is Borehole radius (cm), H_1 is the first head of water established in borehole (cm) , H_2 is the second head of water established in borehole (cm) and C is Shape factor (from Table 2).

One Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$	$K_{fs} = \frac{C_1 \times Q_1}{2\pi H_1^2 + \pi a^2 C_1 + 2\pi\left(\frac{H_1}{a^*}\right)}$ $\Phi_m = \frac{C_1 \times Q_1}{(2\pi H_1^2 + \pi a^2 C_1)a^* + 2\pi H_1}$
One Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$	
Two Head, Combined Reservoir	$Q_1 = \bar{R}_1 \times 35.22$ $Q_2 = \bar{R}_2 \times 35.22$	$G_1 = \frac{H_2 C_1}{\pi(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1))}$ $G_2 = \frac{H_1 C_2}{\pi(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1))}$ $K_{fs} = G_2 Q_2 - G_1 Q_1$ $G_3 = \frac{(2H_2^2 + a^2 C_2)C_1}{2\pi(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1))}$ $G_4 = \frac{(2H_1^2 + a^2 C_1)C_2}{2\pi(2H_1 H_2(H_2 - H_1) + a^2(H_1 C_2 - H_2 C_1))}$ $\Phi_m = G_3 Q_1 - G_4 Q_2$
Two Head, Inner Reservoir	$Q_1 = \bar{R}_1 \times 2.16$ $Q_2 = \bar{R}_2 \times 2.16$	

K from Grain Size Analysis Report

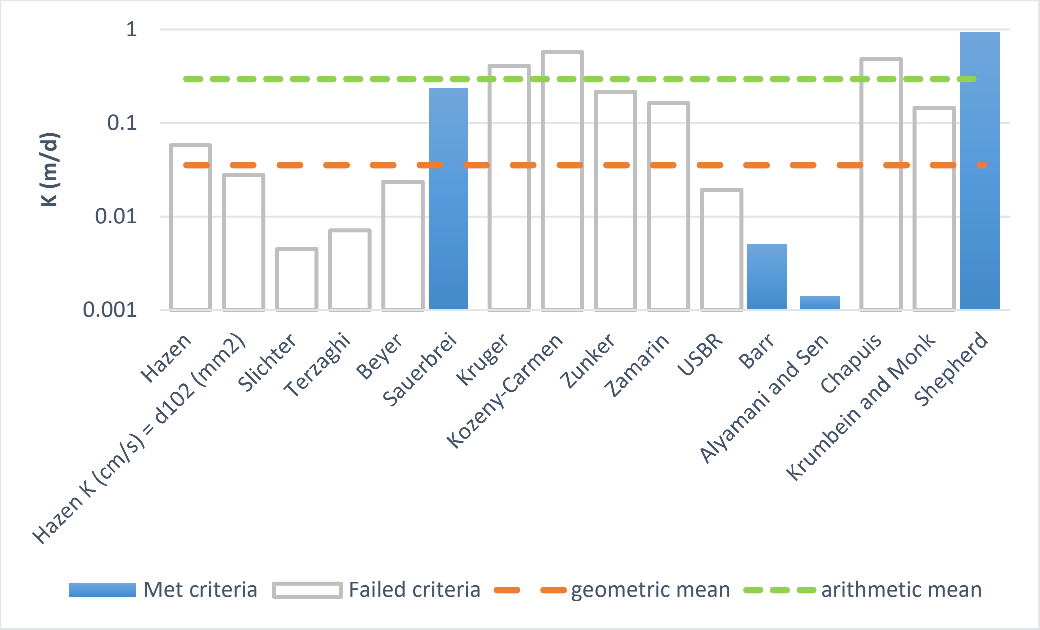
Date: 16-Jun-23

Sample Name: 23-02 (SA5)

Mass Sample (g): 100

T (oC) 20

Poorly sorted silt low in fines



Estimation of Hydraulic Conductivity	cm/s	m/s	m/d	de
Hazen	.669E-04	.669E-06	0.06	
Hazen K (cm/s) = d_{10} (mm)	.321E-04	.321E-06	0.03	
Slichter	.523E-05	.523E-07	0.00	
Terzaghi	.820E-05	.820E-07	0.01	
Beyer	.273E-04	.273E-06	0.02	
Sauerbrei	**.275E-03**	**.275E-05**	**0.24**	
Kruger	.470E-03	.470E-05	0.41	
Kozeny-Carmen	.659E-03	.659E-05	0.57	
Zunker	.249E-03	.249E-05	0.21	
Zamarin	.190E-03	.190E-05	0.16	
USBR	.224E-04	.224E-06	0.02	
Barr	**.591E-05**	**.591E-07**	**0.01**	
Alyamani and Sen	**.165E-05**	**.165E-07**	**0.00**	
Chapuis	.561E-03	.561E-05	0.48	
Krumbein and Monk	.168E-03	.168E-05	0.14	
Shepherd	**.108E-02**	**.108E-04**	**0.94**	
geometric mean	.413E-04	.413E-06	0.04	
arithmetic mean	.342E-03	.342E-05	0.30	

K from Grain Size Analysis Reeport

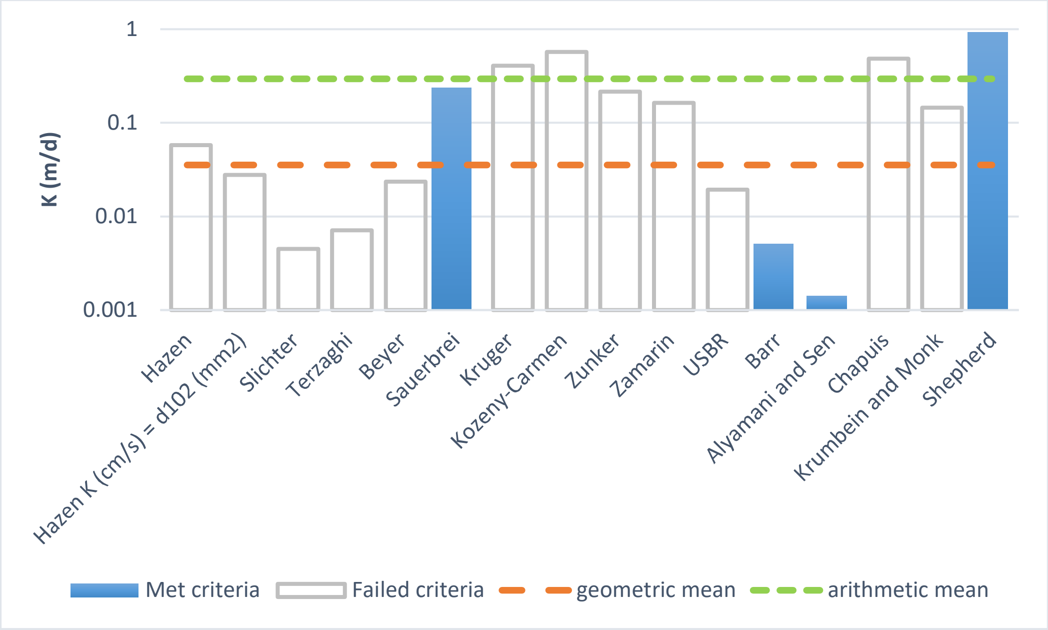
Date: 16-Jun-23

Sample Name: 23-03 (SA3)

Mass Sample (g): 100

T (oC) 20

Poorly sorted silt low in fines



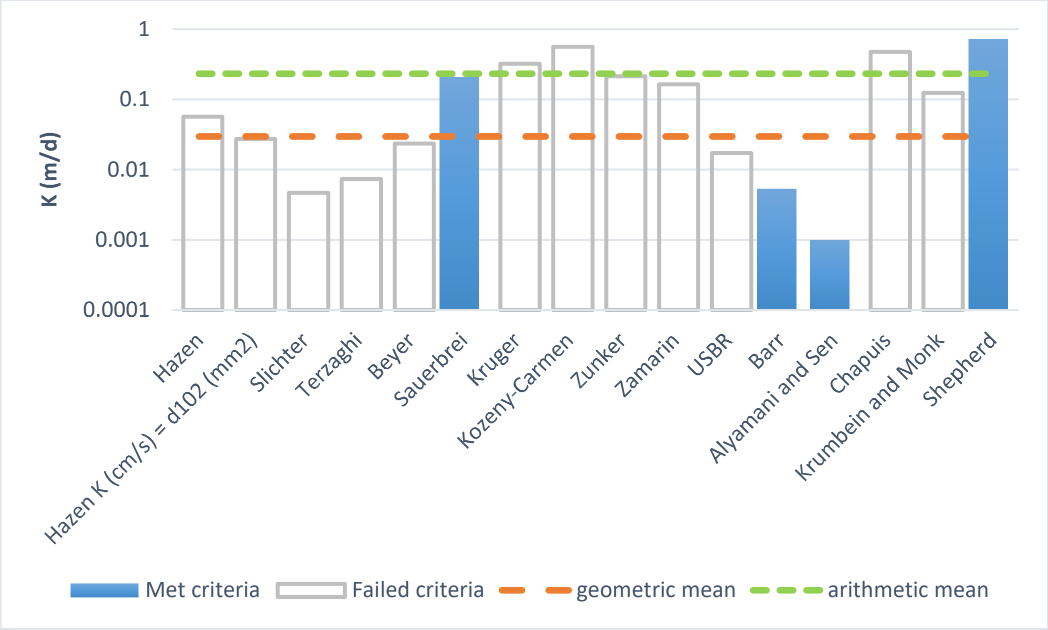
Estimation of Hydraulic Conductivity	cm/s	m/s	m/d	de
Hazen	.669E-04	.669E-06	0.06	
Hazen K (cm/s) = d_{10} (mm)	.321E-04	.321E-06	0.03	
Slichter	.523E-05	.523E-07	0.00	
Terzaghi	.820E-05	.820E-07	0.01	
Beyer	.273E-04	.273E-06	0.02	
Sauerbrei	**.275E-03**	**.275E-05**	**0.24**	
Kruger	.470E-03	.470E-05	0.41	
Kozeny-Carmen	.659E-03	.659E-05	0.57	
Zunker	.249E-03	.249E-05	0.21	
Zamarin	.190E-03	.190E-05	0.16	
USBR	.224E-04	.224E-06	0.02	
Barr	**.591E-05**	**.591E-07**	**0.01**	
Alyamani and Sen	**.165E-05**	**.165E-07**	**0.00**	
Chapuis	.561E-03	.561E-05	0.48	
Krumbein and Monk	.168E-03	.168E-05	0.14	
Shepherd	**.108E-02**	**.108E-04**	**0.94**	
geometric mean	.413E-04	.413E-06	0.04	
arithmetic mean	.342E-03	.342E-05	0.30	

K from Grain Size Analysis Report Date: 16-Jun-23

Sample Name: 23-04 (SA5)

Mass Sample (g): 100 T (oC) 20

Poorly sorted silt with fines



Estimation of Hydraulic Conductivity	cm/s	m/s	m/d	de
Hazen	.656E-04	.656E-06	0.06	
Hazen K (cm/s) = d_{10} (mm)	.315E-04	.315E-06	0.03	
Slichter	.538E-05	.538E-07	0.00	
Terzaghi	.852E-05	.852E-07	0.01	
Beyer	.272E-04	.272E-06	0.02	
Sauerbrei	.241E-03	.241E-05	0.21	
Kruger	.373E-03	.373E-05	0.32	
Kozeny-Carmen	.653E-03	.653E-05	0.56	
Zunker	.248E-03	.248E-05	0.21	
Zamarin	.190E-03	.190E-05	0.16	
USBR	.198E-04	.198E-06	0.02	
Barr	.613E-05	.613E-07	0.01	
Alyamani and Sen	.114E-05	.114E-07	0.00	
Chapuis	.552E-03	.552E-05	0.48	
Krumbein and Monk	.143E-03	.143E-05	0.12	
Shepherd	.832E-03	.832E-05	0.72	
geometric mean	.344E-04	.344E-06	0.03	
arithmetic mean	.270E-03	.270E-05	0.23	

K from Grain Size Analysis Report

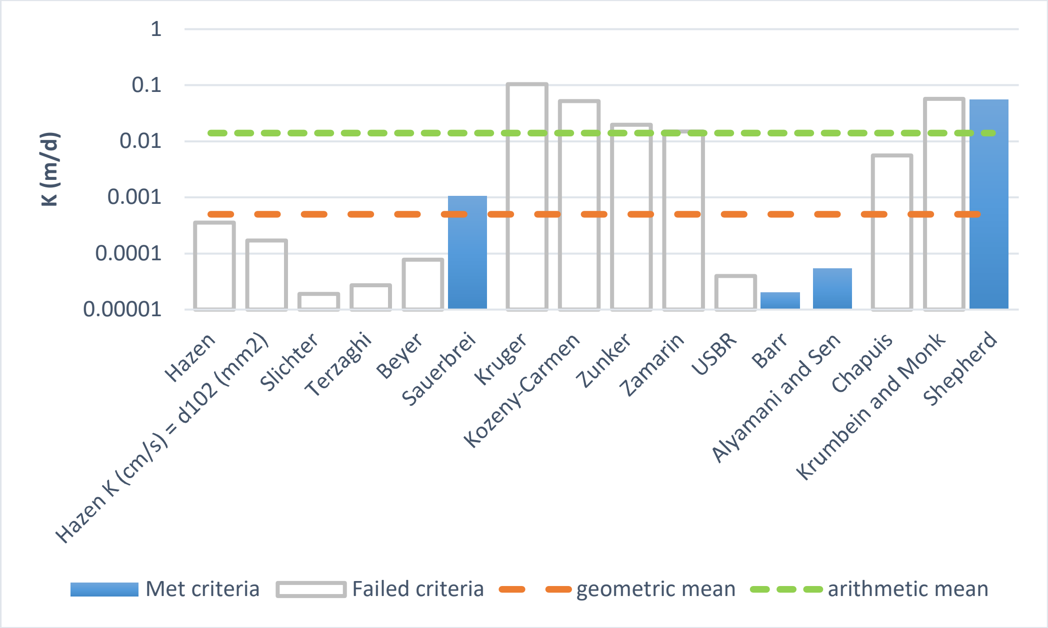
Date: 16-Jun-23

Sample Name: GP23-02

Mass Sample (g): 100

T (oC) 20

Poorly sorted clay with fines



Estimation of Hydraulic Conductivity	cm/s	m/s	m/d	de
Hazen	.412E-06	.412E-08	0.00	
Hazen K (cm/s) = d_{10} (mm)	.197E-06	.197E-08	0.00	
Slichter	.220E-07	.220E-09	0.00	
Terzaghi	.313E-07	.313E-09	0.00	
Beyer	.893E-07	.893E-09	0.00	
Sauerbrei	**.124E-05**	**.124E-07**	**0.00**	
Kruger	.121E-03	.121E-05	0.10	
Kozeny-Carmen	.604E-04	.604E-06	0.05	
Zunker	.228E-04	.228E-06	0.02	
Zamarin	.174E-04	.174E-06	0.02	
USBR	.457E-07	.457E-09	0.00	
Barr	**.235E-07**	**.235E-09**	**0.00**	
Alyamani and Sen	**.623E-07**	**.623E-09**	**0.00**	
Chapuis	.646E-05	.646E-07	0.01	
Krumbein and Monk	.659E-04	.659E-06	0.06	
Shepherd	**.644E-04**	**.644E-06**	**0.06**	
geometric mean	.584E-06	5.84E-09	0.00	
arithmetic mean	.164E-04	1.64E-07	0.01	

K from Grain Size Analysis Report

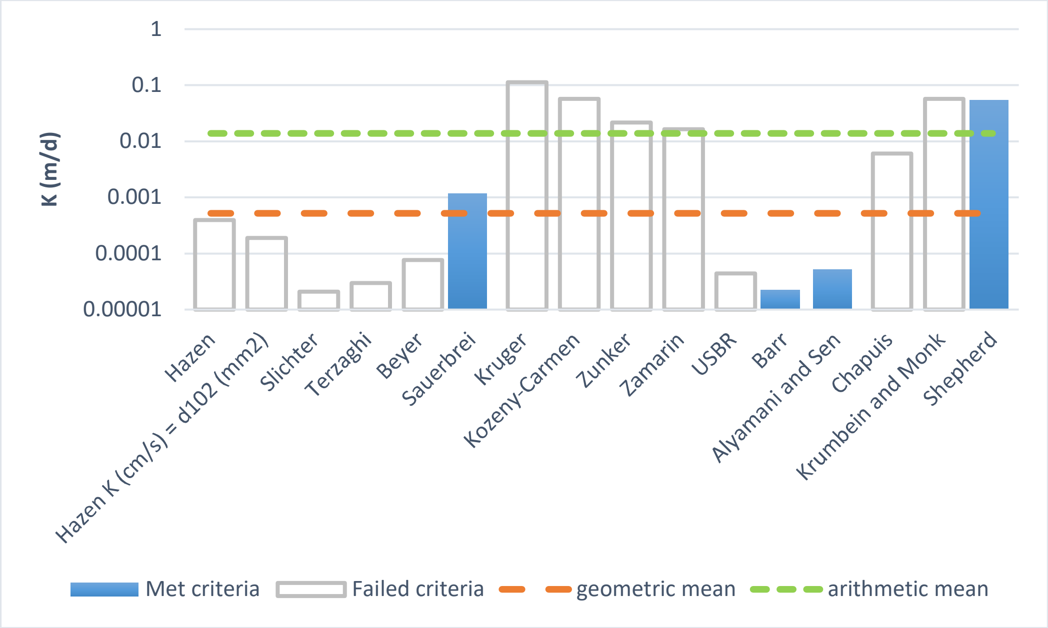
Date: 16-Jun-23

Sample Name: GP23-04

Mass Sample (g): 100

T (oC) 20

Poorly sorted clay with fines



Estimation of Hydraulic Conductivity	cm/s	m/s	m/d	de
Hazen	.454E-06	.454E-08	0.00	
Hazen K (cm/s) = d_{10} (mm)	.218E-06	.218E-08	0.00	
Slichter	.242E-07	.242E-09	0.00	
Terzaghi	.345E-07	.345E-09	0.00	
Beyer	.881E-07	.881E-09	0.00	
Sauerbrei	**.136E-05**	**.136E-07**	**0.00**	
Kruger	.131E-03	.131E-05	0.11	
Kozeny-Carmen	.660E-04	.660E-06	0.06	
Zunker	.249E-04	.249E-06	0.02	
Zamarin	.190E-04	.190E-06	0.02	
USBR	.512E-07	.512E-09	0.00	
Barr	**.260E-07**	**.260E-09**	**0.00**	
Alyamani and Sen	**.608E-07**	**.608E-09**	**0.00**	
Chapuis	.704E-05	.704E-07	0.01	
Krumbein and Monk	.663E-04	.663E-06	0.06	
Shepherd	**.634E-04**	**.634E-06**	**0.05**	
geometric mean	6.08E-07	6.08E-09	0.00	
arithmetic mean	1.62E-05	1.62E-07	0.01	

experience • knowledge • integrity

civil	civil
geotechnical	géotechnique
environmental	environnementale
field services	surveillance de chantier
materials testing	service de laboratoire des matériaux

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