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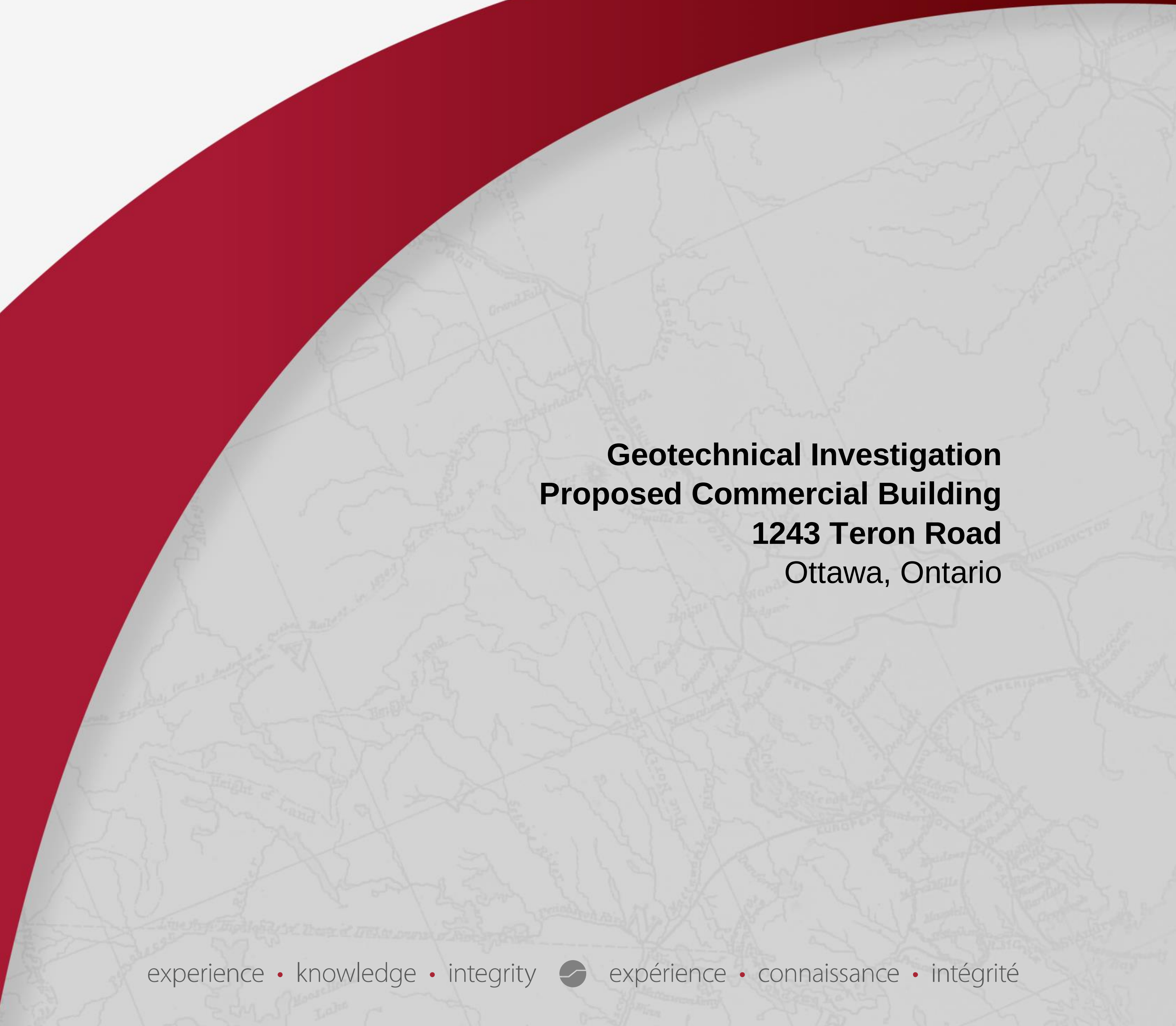

Geotechnical Investigation
Proposed Commercial Building
1243 Teron Road

Ottawa, Ontario

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Submitted to:

Megha Holdings Inc.
1558 Blohm Drive
Ottawa, Ontario
K1G 4R7

Geotechnical Investigation Proposed Commercial Building 1243 Teron Road

Ottawa, Ontario

October 30, 2020
Project: 64742.02

experience • knowledge • integrity

GEMTEC Consulting Engineers and Scientists Limited
32 Steacie Drive
Ottawa, ON, Canada
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October 30, 2020 File: 64742.02 – R06

Megha Holdings Inc.
1558 Blohm Drive
Ottawa, Ontario
K1G 4R7

Attention: Mr. Ramesh Sarna - Director

Re: Geotechnical Investigation
Proposed Commercial Building
1243 Teron Road
Ottawa, Ontario

Enclosed is our geotechnical investigation report for the above noted project. This report was prepared in accordance with the scope of work provided in our proposals dated July 12, 2019, February 24, 2020 and May 8, 2020. This report was prepared by Gregory Davidson, P.Eng. and reviewed by Johnathan A. Cholewa, Ph.D., P.Eng.

G. Davidson

Greg Davidson, P.Eng.
Geotechnical Engineer

Johnathan A. Cholewa, Ph.D., P.Eng.
Geotechnical Engineer

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Enclosures
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out for the proposed commercial building located at 1243 Teron Road in Kanata (Ottawa), Ontario (see Key Plan). 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 factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the project, including construction considerations that could influence design decisions.

This investigation was carried out in general accordance with our proposals dated July 12, 2019, February 24, 2020 and May 8, 2020.

2.0 PROJECT DESCRIPTION AND SITE GEOLOGY

2.1 Project Description

Based on a preliminary drawing prepared by KWC Architects and provided to GEMTEC Consulting Engineers and Scientists Limited (GEMTEC), it is understood that a commercial structure is to be constructed on the (vacant) portion of the property at 1243 Teron Road (i.e., east side of the property). The site is currently vegetated with small to large trees. In addition, there is an existing stormwater management ditch located in the northwest corner of the vacant land.

It is understood that the proposed building will have underside of footing elevation ranging between 81.5 and 85.0 metres (increasing in elevation from the northwest to the southeast) , and a finished floor elevation of about 85.5 metres. Based on the ground surface elevations at the borehole locations, the existing grades at the site range between elevation 84.0 and 90.4 metres, increasing in elevation from northwest to southeast. As such, in the southeast portion of the site, the proposed finished floor elevation will be below existing grade. In the northwest portion of the site, the proposed finished floor elevation will be above the existing grade and grade raising filling will be required.

2.2 Site Geology

Surficial geology maps of the Ottawa area indicate that the north portion of the proposed site is underlain by about 10 to 15 metres of silty clay, and a thin layer of overburden material (2 metres or less) overlying bedrock underlies the south portion of the site. Bedrock geology maps of the area show that the overburden deposits are underlain by Precambrian bedrock of granitic origin. Fill material associated with the development of the adjacent portion of the site should be anticipated.

3.0 METHODOLOGY

3.1 Geotechnical Investigation

The field work for this investigation was carried out between August 15 and 16, 2019 as well as between March 3 and 4, 2020 and June 9 and 10, 2020. During this time, fourteen (14) boreholes numbered 19-1 and 19-3 to 19-12, inclusive, and 20-1 to 20-5, inclusive, were advanced at the site by CCC Geotechnical and Environmental Drilling of Ottawa, Ontario, to depths ranging from about 2.4 to 9.4 metres below existing grade (elevations 78.0 to 87.2 metres, geodetic datum). It should be noted that borehole 19-2 was not advanced at the site due to limited access from existing trees and vegetation.

Standard penetration tests (SPT) were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter split barrel sampler. The bedrock was cored using HQ size coring equipment.

The field work was observed throughout by a member of our engineering staff who directed the drilling operations and logged the samples and boreholes.

Six (6) standpipe piezometers were installed and sealed in the overburden at borehole locations 19-6, 19-12 and 20-2 to 20-5, inclusive, to facilitate groundwater level measurements.

Following completion of the drilling, the soil and bedrock samples were returned to our laboratory for examination by a geotechnical engineer and for classification testing. Four (4) samples of the soil recovered from boreholes 19-3, 19-6, 19-7 and 19-10 were sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

The results of the boreholes are provided on the Record of Borehole sheets in Appendix A. The approximate locations and ground surface elevations of the boreholes are shown on the Borehole Location Plan, Figure 1. The laboratory testing results are provided on the Soils Grading and Plasticity charts in Appendix B. The results of the chemical analysis of soil samples relating to corrosion of buried concrete and steel are provided in Appendix C. Pictures of the recovered bedrock cores are provided in Appendix D.

The borehole locations were selected by GEMTEC and positioned on site relative to existing features. The ground surface elevations at the location of the boreholes were determined using a Trimble R10 global positioning system. The coordinates and elevations of the boreholes are considered to be accurate within the tolerance of the instrument.

3.2 Hydraulic Testing

Hydraulic testing was carried out in the well screen installed in boreholes 20-2 to 20-5, inclusive, in order to estimate the hydraulic conductivity of the bedrock within the anticipated depth of possible excavations and, if possible, to provide an estimate of the potential quantity of water entering future excavations.

The hydraulic testing included falling and rising head tests; however, the falling head test data in borehole 20-2 could not be analyzed due to water overflowing upon slug insertion and a rising head test was not completed in borehole 20-4 due to insufficient recovery during the falling head test. Hydraulic testing was carried out on March 13, 2020 in borehole 20-2 and June 17, 2020 in boreholes 20-3, 20-4 and 20-5. In addition, hydraulic testing in borehole 20-2 included purging the well dry and monitoring the recovery.

The well screens were installed within a surround of filter sand. Above the surround of filter sand, bentonite pellets were used to seal the monitoring well from the soil above. Details of the well screen are provided on the Record of Borehole sheet (Appendix A). The results of the hydraulic testing are provided in Appendix E.

3.3 Subsurface conditions

3.3.1 General

As previously indicated, the soil, bedrock, and groundwater conditions identified in the boreholes are given on the Record of Borehole sheets in Appendix A. The logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. The precision with which subsurface conditions are indicated depends on the method of drilling and excavation, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the test locations may vary from the conditions encountered in the boreholes and test pits. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties.

The soil and bedrock descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and GEMTEC does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. Groundwater conditions may vary seasonally or as a consequence of construction activities in the area.

The following presents an overview of the subsurface conditions encountered in the boreholes advanced during this investigation. It should be noted that the overburden material was not logged in boreholes 20-1, 20-2, 20-3 and 20-5. As such, the following overview of subsurface conditions applies to boreholes 19-1 and 19-3 to 19-12, inclusive, and 20-4, unless otherwise stated.

3.3.2 Topsoil

A surficial layer of topsoil material was encountered at all borehole locations. The topsoil consists of dark brown silty sand with organic material. The thickness of the topsoil ranges from about 50 to 200 millimetres.

3.3.3 Fill Material

Fill material was encountered at all borehole locations below the surficial topsoil material. The fill material is variable across the site but can generally be described as brown silty sand with trace to some clay and gravel. The thickness of the fill material ranges from about 0.4 to 2.0 metres and extends to depths ranging from about 0.6 to 2.1 metres below existing grade.

Standard penetration tests carried out in the fill material gave N values ranging between 10 and 24 blows per 0.3 metres of penetration, which reflects a loose to compact relative density.

The results of grain size distribution testing on a sample of the fill material are provided on the Soils Grading Charts in Appendix B and summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Testing (Fill Material)

Borehole	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
19-3	3	1.5 – 2.1	0	20	42	38

3.4 Weathered Crust

Native deposits of grey to brown silty clay with varying amounts of sand (herein referred to as weathered crust) were encountered at all borehole locations below the fill material at depths ranging from about 0.6 to 2.1 metres below existing grade. The thickness of the weathered crust at boreholes 19-6 to 19-12, inclusive, (where the boreholes were advanced through the weathered crust) ranges from about 1.7 to 5.0 metres and extends to depths ranging from about 2.7 to 5.8 metres below existing grade. Boreholes 19-1, 19-3, 19-4 and 19-5 were terminated within the weathered crust at a depth of about 6.0 metres below existing grade.

Standard penetration tests carried out in the weathered crust gave N values ranging between 3 and 23 blows per 0.3 metres of penetration. Based on our experience with native clays in the Ottawa region, N values of 2 or greater reflect a stiff to very stiff consistency.

The results of grain size distribution testing on samples of the weathered crust are provided on the Soils Grading Charts in Appendix B and summarized in Table 4.2.

Table 4.2 – Summary of Grain Size Distribution Testing (Weathered Crust)

Borehole	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
19-8	2	0.8 – 1.4	0	9	27	64
19-11	6	3.8 – 4.4	0	2	39	59

The results of Atterberg limit testing carried out on samples of the weathered crust are provided on Plasticity Charts in Appendix B and summarized in Table 4.3

Table 4.3 – Summary of Atterberg Limit Testing (Weathered Crust)

Borehole	Sample Number	Sample Depth (metres)	Water Content (%)	LL (%)	PL (%)	PI (%)
19-1	4	2.3 – 2.9	31	40	22	18
19-6	3	1.5 – 2.1	36	48	27	22
19-11	2	0.8 – 1.4	34	45	22	24
19-11	4	2.3 – 2.9	44	45	21	24

Moisture content testing carried out on samples of the weathered crust indicate moisture contents ranging from about 28 to 49 percent.

3.4.1 Glacial Till

Native deposits of glacial till were encountered in boreholes 19-6, 19-7, 19-9, 19-10 and 19-11 below the weathered crust at depths ranging from about 2.7 to 5.8 metres below existing grade. Glacial till is a heterogeneous mixture of all grain sizes; however, at this site the glacial till can generally be described as brown sand and silt with some gravel.

One standard penetration test carried out in the glacial till encountered in 19-11 gave an N value of 16 blows per 0.3 metres of penetration, which reflects a compact relative density.

The results of grain size distribution testing on a sample of the glacial till from borehole 19-11 are provided on the Soils Grading Charts in Appendix B and summarized in Table 4.4.

Table 4.4 – Summary of Grain Size Distribution Testing (Glacial Till)

Borehole	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt and Clay (%)
19-11	8	5.3 – 5.9	11	52	37

Moisture content testing carried out on a sample of the glacial till indicates a moisture content of about 15 percent.

3.5 Auger refusal

Auger refusal was encountered in boreholes 19-6 to 19-12, inclusive, as well as in boreholes 20-1 and 20-2 at depths ranging from about 2.1 to 6.1 metres below existing grade (elevations, 80.3 to 88.2 metres). It should be noted that auger refusal can occur on the bedrock surface or on boulders within the glacial till.

3.6 Bedrock

The bedrock was cored in boreholes 20-1 to 20-5, inclusive, using HQ sized coring equipment from about 2.1 to 6.3 metres below surface grade (elevations 80.0 to 88.2 metres, geodetic datum). The bedrock was cored to depths ranging from about 3.9 to 9.4 metres below ground surface (elevations 78.0 to 86.4 metres, geodetic datum). Pictures of the recovered bedrock cores are provided in Appendix D.

The bedrock consists of grey and pink, faintly to slightly weathered, close to moderately fractured, granitic bedrock with oxidation at the joint fractures. The solid core recovery (SCR) values range from 37 to 100 percent, and the rock quality designation (RQD) values range from 20 to 89 percent. Therefore, the bedrock quality is very poor to good and generally increases in quality with depth.

Four (4) bedrock core samples were tested for unconfined compressive strength and the results are summarized in Table 4.5 below and provided in Appendix D.

Table 4.5 – Unconfined Compressive Strength of Bedrock Core – Boreholes 20-1 and 20-2

Borehole	Sample No.	Depth (metres)	Unconfined Compressive Strength (MPa)
20-1	RC 1	2.4 – 2.6	37
20-2	RC 4	6.0 – 6.2	92

Borehole	Sample No.	Depth (metres)	Unconfined Compressive Strength (MPa)
20-3	RC 1	6.5 – 6.7	199
20-5	RC 2	8.3 – 8.5	50

Based on the unconfined compressive strength test results presented in Table 4.5, the bedrock strength may be classified as medium strong to very strong.

3.7 Groundwater

The groundwater levels were measured on August 23, 2019, March 10, 2020, March 14, 2020, June 17, 2020 and June 24, 2020 and are summarized in Table 4.6.

Table 4.6 – Groundwater Level Observations

Borehole	Depth Below Existing Ground Surface August 23, 2020 (metres)	Depth Below Existing Ground Surface March 10, 2020 (metres)	Depth Below Existing Ground Surface March 14, 2020 (metres)	Depth Below Existing Ground Surface June 17, 2020 (metres)	Depth Below Existing Ground Surface June 24, 2020 (metres)
19-6	2.8	-	-	-	1.57
19-12	Dry	-	-	-	-
20-2	-	0.3 (above ground surface)	0.35 (above ground surface)	-	1.60
20-3	-	-	-	1.12	1.42
20-4	-	-	-	1.04	1.26
20-5	-	-	-	1.31	1.59

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.

Monitoring wells were installed in boreholes 20-3, 20-4 and 20-5 in order to determine groundwater flow directions in the bedrock aquifer as well as assess vertical hydraulic gradients.

Based on the groundwater levels measured in the bedrock monitoring wells on June 24, 2020, the approximate groundwater flow direction is north-northwest. Monitoring wells installed in close proximity, e.g. borehole 19-6 (overburden) / 20-3 (bedrock) and borehole 20-2 (bedrock) / 20-5 (bedrock) are used to estimate vertical hydraulic gradients, summarized in Table 4.7.

Table 4.7 – Vertical Hydraulic Gradients

BH	Well Completion	Screened Interval (metres a.s.l)	Water Level - June 24, 2020 (metres a.s.l)	Vertical Gradients (m/m)
19-6	Overburden	80.25 – 81.77	84.78	0.04 Slightly upward
20-3	Bedrock	78.00 – 79.52	84.87	
20-2	Bedrock	84.05 – 85.57	88.75	-
20-5	Bedrock	80.99 – 84.04	88.76	

1. Metres a.s.l. – metres above sea level.

3.8 Soil and Groundwater Chemistry Relating to Corrosion

The results of chemical testing of soil samples from borehole 19-3, borehole 19-6, borehole 19-7 and borehole 19-10 are provided in Appendix C and summarized in Table 4.8. The results of chemical testing of a groundwater sample from borehole 20-2 are provided in Appendix C and summarized in Table 4.9.

Table 4.8 – Chemical Testing of Soil Samples

BH	Sample	pH	Sulphate Content (micrograms per gram)	Chloride Content (micrograms per gram)	Resistivity (Ohm metres)	Conductivity (mircosiemens per centimetre)
19-3	4	7.8	100	7	57.1	175
19-6	3	8.1	9	7	175	57
19-7	3	7.7	30	8	142	70
19-10	2	7.4	<5	6	265	38

Table 4.9 – Chemical Testing of Groundwater Sample

BH	pH	Sulphate Content (milligrams per litre)	Chloride Content (milligrams per litre)	Resistivity (Ohm metres)
20-2	7.8	17	5	32.1

3.9 Hydraulic Testing Results

The results of the hydraulic testing carried out in the well screens are provided in Appendix E. A summary of the recovery measurements made during the hydraulic testing carried out by introducing a slug into the well screen is provided in Table 4.10.

Table 4.10 – Summary of Falling Head Test Results

Borehole	Geological Material Tested	Static Groundwater Depth[1] (metres bgs)	Slug Displacement Volume (metres)	Initial Groundwater Level Increase[2] (metres)	Recovery Time (minutes)	Recovery (percent)
20-3	Bedrock	1.12	0.45	2.01	5	99
20-4	Bedrock	1.04	0.60	1.73	35	22
20-5	Bedrock	1.31	0.60	0.60	35	95

1. Static groundwater depth measured to be 0.40 metres above ground surface
2. Measured 30 seconds following slug removal

A summary of the recovery measurements made during the hydraulic testing carried out by introducing a slug into the well screen is provided in Table 4.11.

Table 4.11 – Summary of Rising Head Test Results

Borehole	Geological Material Tested	Static Groundwater Depth[1] (metres bgs)	Slug Displacement Volume (metres)	Initial Groundwater Level Increase[2] (metres)	Recovery Time (minutes)	Recovery (percent)
20-2	Bedrock	-0.40	0.60	0.56	10	100
20-2 (1)	Bedrock	-0.40	-	6.0	30	99
20-3	Bedrock	1.12	0.45	0.37	5	99
20-5	Bedrock	1.31	0.60	1.70	25	95

1. Borehole 20-2 purged dry and recovery monitored.

The hydraulic conductivity calculated from the hydraulic test results are provided in Table 4.12.

Table 4.12 – Calculated Hydraulic Conductivity

Borehole	Geological Material Monitored	Calculated Hydraulic Conductivity, Falling Head Test (m/s)[1,2]	Calculated Hydraulic Conductivity – Rising Head Test (m/s)[1,2]
20-2	Bedrock	-	8×10^{-6} / 5×10^{-6} (3)
20-3	Bedrock	1×10^{-5}	9×10^{-6}
20-4	Bedrock	6×10^{-8}	-
20-5	Bedrock	8×10^{-7}	1×10^{-6}

Notes:

1. The hydraulic conductivities were calculated using the Hvorslev analysis.
2. Displacement volume of slug used in analysis (0.45 metres for borehole 20-3 and 0.60 metres for boreholes 20-2, 20-4 and 20-5).
3. Borehole 20-2 purged dry and recovery monitored.

The bedrock hydraulic conductivity, calculated from hydraulic testing completed in monitoring wells 20-2, 20-3, 20-4 and 20-5 ranges from 1×10^{-5} to 6×10^{-8} m/s. The calculated hydraulic conductivity is within literature values (Freeze and Cherry, 1979) for granitic bedrock, which has a hydraulic conductivity ranging from 10^{-4} to 10^{-8} m/s. Based on the variability of calculated hydraulic conductivity between boreholes, the hydraulic conductivity is expected to vary throughout the bedrock formation based on the presence, or absence, of discontinuities and fractures.

4.0 GEOTECHNICAL GUIDELINES AND RECOMMENDATIONS

4.1 General

This section of the report provides engineering guidelines on the geotechnical design aspects of the project based on our interpretation of the boreholes advanced as part of this investigation and the project requirements. It is stressed that the information in the following sections is provided for the guidance of the designers and is intended for this project only. Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site from materials from offsite sources are outside the terms of reference for this report and have not been investigated or addressed.

4.2 Proposed Commercial Building

4.2.1 Overburden Excavation

Based on the boreholes advanced in the vicinity of the proposed building, the excavations for the proposed buildings will be carried out mostly through topsoil, fill material and silty clay and/or glacial till. The sides of the excavation 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 fill material at this site can be classified as Type 3 soil and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter.

In the event that a granular pad is necessary below the foundations, the excavations should be sized to accommodate a pad of imported granular material which extends at least 0.5 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

4.2.2 Bedrock Excavation

For a finished floor elevation of 85.5 metres, it is anticipated that bedrock removal will be required in the area of boreholes 19-8, 19-9, 19-10, 19-12, 20-1, 20-2, and 20-05.

Localized bedrock removal at this site could be carried out using (a) drill and blasting, (b) hoe ramming techniques in conjunction with line drilling on close centres or (c) a combination of both. Provided that good bedrock excavation techniques are used, the bedrock could be excavated using near vertical side walls. Any loose bedrock should be scaled from the sides of the excavation.

It is noted that the Precambrian bedrock is known to be abrasive and hard, and significant equipment wear should be expected.

Any blasting should be carried out under the supervision of a blasting specialist engineer. As a guideline for blasting, the peak vibration limits suggested at the nearest structure or service are provided in Table 5.1.

Table 5.1 – Peak Vibration Limits

Frequency of Vibration (Hz)	Vibration Limits (millimetres/second)
<10	5
10 to 40	5 to 50 (interpolated)
>40	50

It is pointed out that the limits provided in Table 5.1, although conservative, were established to prevent damage to existing buildings and services in good condition; more stringent criteria may be required to prevent damage to freshly placed (uncured) concrete or vibration sensitive equipment or utilities. Monitoring of the blasting should be carried out to ensure that the blasting meets the limiting vibration criteria. Pre-construction condition surveys of the nearby structures and existing buried services are considered essential. The effects due to vibration from blasting can be controlled by limiting the size and amount of charge, using delayed detonation techniques, and the like. To reduce the effects of vibration on nearby services, we suggest that the separation distance between any blasting and existing underground services be at least 6 metres. Any bedrock removal within these limits could be carried out using hoe ramming techniques in conjunction with line drilling on close centres. It is noted that the cost of bedrock removal generally increases the closer the bedrock removal is to any existing structures or services.

As an alternative to blasting, bedrock removal could be carried out using hoe ramming techniques in conjunction with line drilling on close centres. For the bedrock at this site, it is suggested that allowance be made for line drilling 75 to 100 millimetre diameter holes on 200 to 300 millimetre centres. It should be noted that based on bedrock coring work undertaken at this site, significant effort may be required to break the bedrock using this method.

The vibration effects of hoe ramming are usually minor and localized. Monitoring of the hoe ramming could be carried out, at least initially, to measure the vibrations to ensure that they are below the acceptable threshold value. Provided that good bedrock excavation techniques are used, the bedrock could be excavated using vertical side walls. Any loose rock should be scaled from the sides of the excavation.

The bedrock may contain numerous irregular discontinuities. As such, significant overbreak and underbreak should be expected in any bedrock removal. The bedrock below founding level will likely break at a horizontal bedding plane below the design depth of the footings, which may necessitate thickening of the footings and/or lowering of the footings.

4.2.3 Basal Heaving

It is noted that there is potential for basal heaving where the piezometric pressure in the bedrock exceeds the weight of the soil between the base of the excavation and surface of the bedrock. Heaving of the floor of the excavation could cause disturbance of the subgrade soils and lead to a rapid increase in groundwater inflow. Based on the results of the investigation, there is a risk of basal heaving where base of the excavation is located in close proximity to the surface of the pressurized bedrock (i.e., in the area of boreholes 19-7, 19-11, and, depending on the depth of excavation, in the area of borehole 19-6).

For excavation below elevation 84 metres in the area of boreholes 19-7, 19-11, and 20-4, we recommend that the groundwater level in the bedrock be lowered prior to excavation in order to increase the factor of safety against basal heaving to an acceptable level. If an excavation below elevation 84 metres is carried out during the spring, groundwater level lowering in the bedrock will also be required in the area of boreholes 19-6 and 20-3. This could be achieved by pumping from deep wells installed in the bedrock. Alternatively, the excavation operations could be planned in such a way so that bedrock excavation to founding level in the area of boreholes 19-8, 19-9, 19-10, 19-12, 20-1, 20-2, and 20-5 is carried out prior to overburden excavation at boreholes 19-7, 19-11, and 20-4 (i.e., it is anticipated that dewatering of the bedrock excavation in the area of boreholes 19-8, 19-9, 19-10, 19-12, 20-1, 20-2, and 20-5 will be effective at depressurizing the underlying bedrock at boreholes 19-7, 19-11, and 20-4).

It is noted that no information is available on the long-term groundwater levels throughout the year; however, based on the measured groundwater levels to date, it is expected that the groundwater levels will be lower during drier periods of the year, such as summer and early fall, relative to the groundwater levels measured in March 2020. It is noted that, the water levels measured in June 2020 in borehole 20-2 were much lower than the March 2020 water levels. We recommend that the groundwater level in the well screens installed as part of this investigation be measured periodically throughout the year to determine the magnitude of seasonal groundwater fluctuations in the overburden and bedrock. This information could be used for construction planning and developing dewatering/depressurization methodologies.

4.2.4 Construction Dewatering

In order to reduce groundwater pumping requirements, we recommend that the excavation be planned for drier periods of the year, such as summer and early fall.

Groundwater inflow from the overburden deposits and bedrock into the excavations could be controlled by pumping from filtered sumps within the excavations. It is not expected that short term pumping during excavation will have any significant affect on nearby structures and services.

Based on the measured artesian conditions in bedrock monitoring well MW20-2, excavation within bedrock may result in significant groundwater inflows. Further, in order to increase the factor of safety against basal heaving to an acceptable level, groundwater level lowering in the bedrock will likely be required prior to excavation in the area of boreholes 19-7, 19-11, and 20-4. As such, the groundwater inflow for this project may exceed 400,000 litres per day. Furthermore, based on the anticipated size of the open excavation, significant groundwater/stormwater pumping may be required following the spring freshet and significant rainfall events.

As such, a Category 3 Permit To Take Water (PTTW) is recommended for short-term construction. The Category 3 PTTW application is submitted to the Ministry of the Environment, Conservation and Parks (MECP) for review and requires a hydrogeological report. In addition, a long-term PTTW may be required and if groundwater is discharged to the City of Ottawa storm sewer; a sewer use agreement should be obtained from the City of Ottawa Sewer Use Office. It is noted that if the groundwater does not meet the water quality limits for discharge to the City of Ottawa storm sewer, an Environmental Compliance Approval (ECA) may also be 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.

4.2.5 Footing Design

Based on the results of the current investigation, the proposed structure could be founded on conventional footings bearing on or within native, undisturbed weathered crust, glacial till or bedrock.

In areas where subexcavation of disturbed material 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 B Type II and should be compacted in maximum 200 millimetre thick lifts to at least 95 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.

Spread footing foundations should be sized using the net geotechnical reactions at Serviceability Limit State (SLS) and factored net geotechnical resistances at Ultimate Limit States (ULS) provided in Table 5.2.

Table 5.2 – Foundation Bearing Values

Subgrade Material	Net Geotechnical Reaction at Serviceability Limit State[1] (kilopascals)	Factored Net Geotechnical Resistance at Ultimate Limit State[2] (kilopascals)
Native, Undisturbed Silty Clay (Weathered Crust)	100	250
Native Undisturbed Glacial Till	120	300
Compacted Engineered Fill (overlying bedrock)	250	400
Sound Bedrock	-	1000

Notes:

1. Provided that all loose or disturbed soil is removed from the bearing surfaces and the engineered fill material is prepared as described above, the post construction total and differential settlement of the footings at SLS should be less than 25 and 20 millimetres respectively. Settlement of footings founded on the bedrock is negligible.

2. Determined using a bearing resistance factor of 0.5 (Canadian Foundation Engineering Manual, Table 8.1, 4th edition, CGS, 2006).

The building will likely be founded on variable subgrade materials, transitioning from weathered crust to bedrock. To reduce the potential for cracking in the foundation walls where the footings transition between different subgrade materials, we suggest that the foundation walls be reinforced with two 15M bars (both top and bottom) for a distance of 3 metres on both sides of the transition.

4.2.6 Seismic Design of Proposed Structures

Based on the blow counts and correlated shear strength values obtained as part of this investigation and Table 4.1.8.4.A of the Ontario Building Code, 2012, Site Class D should be used for the seismic design of the structure. It is pointed out that based on available shear wave velocity mapping, the site could potentially be classified as Site Class C; however, site specific testing would be required to confirm this opinion. Multi-Channel Analysis of Surface Waves (MASW), a non-intrusive geophysical test method could be considered for this purpose.

4.2.7 Frost Protection of the Foundations and Slab

All exterior footings in unheated portions of the proposed structures or slabs should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated, unheated exterior footings adjacent to surfaces that are cleaned of snow cover during the winter months should be provided with a minimum of 1.8 metres of earth cover. The required depth of frost protection can be reduced by the thickness of any engineered fill beneath the foundations. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

4.2.8 Foundation Walls

4.2.8.1 Backfill

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 OPSS Granular B Type I or II requirements.

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. Light, walk behind compaction equipment should be used next to foundation walls to avoid excessive compaction induced stress on the foundation walls. 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.

Where areas of hard surfacing (pavement etc.) 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 material to reduce the effects of differential frost heaving. 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 1 horizontal to 1 vertical, or flatter.

If any areas of the building are to remain unheated during the winter period, thermal protection of the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

4.2.8.2 Drainage

A conventional, perforated perimeter drain should be provided at founding level and either drained by gravity to the storm sewer, or connected to a sump pit equipped with a pump to discharge the water to a storm sewer.

4.2.8.3 Lateral Earth Pressure

The static at rest thrust (P_o) acting on the walls should be calculated using the following formula:

$$P_o = 0.5\ K_o\ \gamma\ H^2$$

where;

- P_o: Static at rest thrust component (kilonewtons);
- γ: Moist material unit weight (kN/m^3);
- K_o: “At Rest” earth pressure coefficient;
- H: Wall height (metres).

Seismic shaking can increase the forces on the retaining wall. The total at rest thrust acting on the wall (P_{oe}) during a seismic event is composed of a static component (P_o) and a dynamic component (P_e), that is:

$$P_{oe} = P_o + P_e$$

The dynamic at rest thrust component (P_e), which acts only during seismic loading conditions, should be calculated using the following formula:

$$P_e = 0.5\ (K_{oe} - K_o)\ \gamma\ H^2$$

where;

- P_e: Dynamic at rest thrust component (kilonewtons);

- γ: Moist material unit weight (kN/m^3);
- K_o: “At Rest” earth pressure coefficient;
- K_{oe}: Dynamic at rest earth pressure coefficient;
- H: Wall height (metres).

The static thrust component (P_o) acts at a point located H/3 above the base of the wall. During seismic shaking, the dynamic at rest thrust component (P_e) acts at a point located about 0.6H above the base of the wall.

For design purposes, the soil parameters provided in Table 5.2 can be used to calculate the at rest thrust components acting on the wall.

Table 5.3 – Summary of Soil Parameters for At Rest Wall

Parameter	OPSS Granular B Type I	OPSS Granular B Type II
Material Unit Weight, γ (kN/m^3)	21	22
Estimated Friction Angle (degrees)	34	38
“At Rest” Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.44	0.38
Dynamic At Rest Earth Pressure Coefficient, K_{oe}, assuming horizontal backfill behind the structure	0.57[1]	0.49[1]
Passive Soil Pressure Coefficient K_p, assuming horizontal backfill in front of the structure	3.07	3.57

Notes:

1) According to the 2015 Ontario Building Code, the peak ground acceleration (PGA) for Ottawa is 0.32 for firm ground conditions (i.e., for Site Class C). For this particular site, the corrected PGA can be taken as 0.37 (Site Class D). The dynamic at rest earth pressure coefficient was calculated using the method suggested by Mononobe and Okabe, assuming a horizontal seismic coefficient, k_h, of 0.37 (taken as the corrected PGA) and assuming that the vertical seismic coefficient, k_v, is zero.

Where conditions dictate, allowance should be made in the structural design of the walls for active loads due to ground supported vehicles/equipment. For example, the horizontal active load due to a uniform, vertical live load adjacent to the foundation wall could be determined using a horizontal earth pressure coefficient, K_o, times the vertical live load. The effects of other vertical loads (point loads, line loads, etc.) adjacent to or near the retaining walls could be provided, if required.

4.2.9 Concrete Floor Slab

4.2.9.1 Slab Support

Based on the results of the investigation, the area in the vicinity of the buildings is generally underlain by topsoil, fill material and native overburden deposits. The existing topsoil and fill material should be removed from the slab on grade areas, such as the fill material encountered in the area of boreholes 19-4 and 19-5.

The grade below the concrete slabs on grade could be raised, where necessary, with granular material meeting OPSS requirements for Granular B Type I or II (i.e., in the northwest portion of the site, assuming a finished floor elevation of 85.5 metres).

Where top of the floor slab is located above original grade at the site, the base for the floor slab should consist of at least 150 millimetres of OPSS Granular A. Where the top of the floor slab is located below original grade at the site, the base of the floor slab should consist of at least 300 millimetres of 19 millimetre clear crushed stone, with a non-woven geotextile meeting OPSS 1860 Class I requirements wherever the clear stone will be in contact with the native soils. The OPSS Granular A should be compacted in maximum 150 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Nominal compaction of the clear stone with at least 2 passes of a diesel plate compactor is recommended to consolidate the material into place.

4.2.9.2 Underfloor Drainage

Where the surface of the floor slab is below the original grades at the site, we recommend that an underfloor drainage system be installed below the slab. The underfloor drainage should consist of regularly spaced, 100 millimetre diameter perforated plastic pipes installed within the 300 millimetre thick clear stone base layer at a spacing of 10 metres. The clear stone base and drains should outlet by gravity to a suitable location. Additional drains may be required in high flow areas or springs, if encountered during construction.

An estimate of long-term groundwater inflow into the underfloor drainage layer is provided in Section 4.7.

4.2.9.3 Slab Uplift Pressure

Section 4.2.3 of this report discusses the risk of basal heave during construction in areas where the piezometric pressure in the bedrock exceeds the weight of the soil between the base of the excavation and surface of the bedrock. In order to mitigate the risk of basal heave during construction, temporary groundwater level lowering in the bedrock (i.e., reduce the piezometric pressure in the bedrock) is recommended.

Similarly, there is a risk of post-construction slab uplift in areas where the piezometric pressure in the bedrock exceeds the weight of the native soil, granular base, and concrete between slab level and the bedrock surface. In order to increase the factor of safety against post-construction slab uplift to an acceptable level, it will be necessary to permanently depressurize the underlying bedrock in the area of boreholes 19-11 and 20-4. The underfloor drainage layer will likely be effective at permanently depressurizing the bedrock in the area of boreholes 19-11 and 20-4.

4.2.9.4 Curing

The floor slab should be wet cured to minimize shrinkage cracking and slab curling. The slab should be saw cut to about 1/3 the thickness of the slab as soon as curing of the concrete permits, in order to minimize shrinkage cracks.

4.2.9.5 Moisture Protection

Proper moisture protection with a vapour retarder should be used for the slabs where the floor will be covered by moisture sensitive flooring material or where moisture sensitive equipment, products or environments will exist. The “Guide for Concrete Floor and Slab Construction”, ACI 302.1R-04 should be considered for the design and construction of vapour retarders below the slabs.

4.3 Retaining Walls

4.3.1 General

The following general geotechnical guidelines are provided for the design and construction of retaining walls at this site:

- Excavation side slopes of 1 horizontal to 1 vertical are recommended within the overburden soil. Excavation within bedrock could be carried our using near vertical side slopes.
- The area required for the excavation will be dependent on the width of the cantilever (heel slab) footing for the wall. It is anticipated that a working area of at least 1.0 metre on the inside edge of the footing would be required to allow for forming/construction. The excavation could then be sloped upwards from this point as per the above recommendations.
- The cantilever wall footing should bear on the native undisturbed soil or bedrock surface. Any topsoil, fill and organic material, or loose/fractured bedrock should be removed from the bearing surfaces prior to placing the formwork for the footings.
- If required, a levelling layer of compacted granular material meeting OPSS requirements for Granular A could be placed below the foundations. The levelling layer should be compacted to at least 98 percent of the standard Proctor dry density value. In areas where the subgrade surface is undulating significantly (i.e. bedrock surface), OPSS Granular B

Type II could be used to provide a level surface. The subbase should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor dry density value. Any granular material below the footing should extend at least 0.3 metres beyond the edges of the footing and slope downwards from this point at 1 horizontal to 1 vertical, or flatter.

- To provide drainage and avoid frost adhesion and possible horizontal frost heaving which could occur behind the wall causing the wall to be pushed or rotated outward, the wall should be backfilled with imported, free-draining, non-frost susceptible granular material meeting OPSS Granular B Type I or II requirements. In order to reduce the earth pressures acting on the back of the wall, consideration could be given to backfilling the wall, or at least a portion of the wall, with lightweight fill. Additional information on the use of lightweight fill as backfill could be provided as the design progresses.
- The retaining walls should be provided with at least 1.8 metres of earth cover for frost protection purposes. Where less than the required depth of soil cover can be provided, the retaining walls can be protected from frost by using a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.
- The non-frost susceptible backfill material should extend at least 1.8 metres horizontally outward from the back of the retaining wall. The backfill should be placed in maximum 200 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.
- To prevent migration of the imported backfill through the weep holes in the concrete wall, and to prevent clogging of the free draining backfill material from migration of the adjacent material, the backfill should be completely wrapped with a nonwoven geotextile.
- Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed retaining wall, 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 materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the bottom of the excavation or 1.8 metres below finished grade, whichever is less, to the underside of the granular base/subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.
- Where future landscaped areas will exist next to the proposed structure 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.
- As a minimum, a perforated drain with a surround of clear crushed stone should be installed along the base of the retaining wall, below finished grade. The drains should outlet by gravity to the ends of the wall and to conduits which pass through the wall to outlet on the toe side of the wall. To avoid loss of sandy backfill into the voids in the clear

stone (and possible post construction settlement of the ground around the wall), a nonwoven geotextile should be placed between the clear stone and any sand backfill material. Perforated and wall drains are not recommended for the retaining walls surrounding the stormwater detention facility. These walls should be designed to resist hydrostatic pressures in the backfill.

- Table 5.3 provides bearing values for the design of the wall foundations.
- A 300 millimetre thick drainage layer of 19 millimetre OPSS clear stone Type I or II should be placed directly behind the wall and wrapped with OPSS Class II nonwoven geotextile. This drainage layer should be hydraulically connected to a 100 millimetre diameter perforated pipe placed near the base of the wall. The drainage pipe should have outlets spaced at not more than 10 metres on centre along the length of the wall. A clear stone drainage layer and perforated pipes are not recommended for retaining walls surrounding the stormwater detention facility. These walls should be designed to resist hydrostatic pressures in the backfill.
- The earth pressure parameters provided in Section 4.2.8.3 could also be used for design of the retaining walls.
- Foundation bearing values for the proposed retaining walls should refer to Table 5.2 above.

4.3.2 Sliding Resistance

For preliminary design purposes, the resistance to sliding of the retaining walls could be calculated using an unfactored interface friction angle of 24 degrees and a friction coefficient of 0.45, assuming that the footings are founded directly on native silty clay; however, if the footings are founded on a pad of compacted granular material or directly on bedrock, the unfactored interface friction angle could be increased to 30 degrees with a friction coefficient of 0.58.

4.3.3 Global Stability

Slope stability analyses were carried out in order to determine the factor of safety against global stability during static and seismic loading conditions. The global stability analyses were carried out using SLIDE, a state of the art, two-dimensional limit equilibrium slope stability program.

The soil conditions and strength parameters used in the stability analyses were based, in part, on the results of the boreholes advanced at the site. In an attempt to model seismic loading conditions, a pseudo-static slope stability analysis was carried out using a seismic coefficient (k_h) of 0.18.

As a conservative approach, the tallest proposed retaining wall geometry was analyzed:

- An exposed wall height of about 4.33 metres;
- A base of 3.3 metres in length; and,

- A wall founded on silty clay (weathered crust) above glacial till overlying bedrock. As a conservative approach, we have assumed that the silty clay is fully saturated.

The input parameters used for the analysis and results for both static and seismic loading conditions are provided on the Slide output Figures F1 and F2 provided in Appendix F.

Based on the results of the analyses, the proposed retaining wall has a factor of safety against global instability of at least 1.5 and 1.1 for both static and seismic loading conditions, respectively.

4.4 Existing Stormwater Management Ditch

There is an existing stormwater management ditch located in the northwest corner that extends towards the south side of the vacant parcel of land. The existing ditch has a maximum height of about 1.6 metres and a slope of about 3 horizontal to 1 vertical. We recommend that the proposed construction (i.e., parking lot etc.) not encroach within 3 metres of the top (crest) of the existing ditch.

4.5 Site Services

4.5.1 Excavation

Based on the available subsurface information, the excavations for the services within the site will be carried out through topsoil, fill material, silty clay, glacial till and possibly bedrock.

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 soils at this site can be classified as Type 3 soils. Therefore, for design purposes, allowance should be made for 1 horizontal to 1 vertical, or flatter, excavation slopes within the native soils at this site. As an alternative to sloping the excavations, all service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

Some disturbance and loosening of the subgrade materials could occur, and allowance should be made for subexcavation and additional pipe bedding (sub-bedding) material, as discussed later in this report.

If bedrock excavation is required for any services at this site, excavation of bedrock should be carried out as described in Section 5.2.2.

4.5.2 Groundwater Pumping

Our comments on construction dewatering provided in Sections 4.2.3 and 4.2.4 also apply to the excavations for site services.

4.5.3 Pipe Bedding and Cover

The bedding for the sanitary sewers, storm sewers and watermains should be in accordance with OPSD 802.010/802.013 and 802.031/802.033 for flexible and rigid pipes in earth and bedrock excavations, respectively. The pipe bedding should consist of at least 150 millimetres of well graded crushed stone meeting OPSS requirements for 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, it is suggested that any granular materials used in the service trenches be composed of virgin (i.e., not recycled) material only.

Allowance should be made for subexcavation of any existing fill, organic deposits, or disturbed material encountered at subgrade level.

Where bedrock is encountered at subgrade level, allowance should be made for additional bedding material due to possibility of overbreak of the bedrock below subgrade level.

Cover material, from pipe spring line to at least 300 millimetres above the top of the pipe, should consist of granular material, such as OPSS Granular A.

The use of clear crushed stone should not be permitted for the installation of site services, since it could exacerbate groundwater lowering of the overburden materials due to “French Drain” effects.

The bedding and cover materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density using suitable vibratory compaction equipment.

4.5.4 Trench Backfill

The general backfilling procedures should be carried out in a manner that is compatible with the future use of the area above the service trenches.

In areas where the service trench will be located below or in close proximity to existing or future roadway/parking lot areas, acceptable native materials should be used as backfill between the roadway/parking lot 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 section of roadway/parking lots. Where native backfill is used, it should match the native 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. The depth of frost penetration in areas that are kept clear of snow and where trench backfill consists of broadly graded shattered rock fill or earth fill is expected to be about 1.8 metres. It is our experience, however, that the frost penetration can be as much as 2.4 metres when the trench backfill consists solely of relatively open graded rock fill. Where cover requirements are not practicable, the pipes could be protected from frost using a combination of earth cover and insulation. Further details regarding insulation could be provided, if required.

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 from the trench. If blast rock is used as backfill within the service trench, it should be mostly 300 millimetres, or smaller, in size and should be well graded. To prevent ingress of fine material into voids in the blast rock, the upper surface of the blast rock should be covered with a thin layer of well graded crushed stone (e.g. OPSS Granular B Type II).

To minimize future settlement of the backfill and achieve an acceptable subgrade for the roadways, parking lots curbs, etc., the trench backfill should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. Rock fill should be placed in maximum 500 millimetre thick lifts and compacted with a large drum roller, the haulage and spreading equipment, or a combination of both. 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.

Most of the overburden deposits at this site are sensitive to changes in moisture content due to percipitation. Depending on the weather conditions at time of construction, the specified densities may not likely be possible to achieve and, as a consequence, some settlement of these backfill materials could occur. Consideration could be 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 compaction.
- Reuse any wet materials in the lower part of the trenches and make provision to defer final paving of any roadways for 6 months, or longer, to allow some the trench backfill settlement to occur and thereby improve the final roadway appearance.
- Reuse any wet materials outside hard surfaced areas and where post construction settlement is less of a concern (such as landscaped areas).

4.5.5 Seepage Barriers

The granular bedding in the service trench could act as a “French Drain”, which could promote groundwater lowering. As such, we suggest that seepage barriers be installed along the service trenches at strategic locations at a horizontal spacing of about 100 metres and where the property meets Teron Road. 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 silty clay. The silty clay should be compacted in maximum 300 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. The locations of the seepage barriers could be provided as the design progresses.

4.6 Access Roadway/Parking Lot Areas

4.6.1 Subgrade Preparation

In preparation for access roadway/parking lot construction at this site, all surficial topsoil, fill material and any soft, wet or deleterious materials should be removed from the proposed roadway areas.

Prior to placing granular material for the internal roads, the exposed subgrade should be inspected and approved by geotechnical personnel. Any soft areas should be subexcavated and replaced with suitable (dry) earth borrow or well shattered and graded rock fill material that is frost compatible with the materials exposed on the sides of the area of subexcavation.

Similarly, should it be necessary to raise the roadway/parking lot grades at this site, material which meets OPSS specifications for Select Subgrade Material, Earth Borrow or well shattered and graded rock fill material may 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. Rock fill should also be placed in maximum 500 millimetre thick lifts and suitably compacted either with a large drum roller, the haulage and spreading equipment, or a combination of both.

Truck traffic should be avoided on the native soil subgrade or the trench backfill within the roadways/parking lot areas especially under wet conditions.

4.6.2 Pavement Structure

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

- 80 millimetres of hot mix asphaltic concrete (Superpave 12.5 (Traffic Level B), placed in two (2) 40 millimetre layers over;
- 150 millimetres of OPSS Granular A base over;
- 300 millimetres of OPSS Granular B, Type II subbase over,
- 300 millimetres of OPSS Granular O over,
- Non-woven geotextile separator meeting OPSS 1860 Class II

For parking areas and access roadways to be used by heavy truck traffic the suggested minimum pavement structure is:

- 100 millimetres of hot mix asphaltic concrete (40 millimetres of Superpave 12.5 (Traffic Level B) over 60 millimetres of Superpave 19.0 (Traffic Level B)), over;
- 150 millimetres of OPSS Granular A base over;
- 450 millimetres of OPSS Granular B, Type II subbase over,
- 300 millimetres of OPSS Granular O over,
- Non-woven geotextile separator meeting OPSS 1860 Class II

If bedrock is encountered at subgrade level, it may be possible to reduce the granular subbase thickness provided above to 150 millimetres.

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. The adequacy of the design pavement thicknesses should be assessed by geotechnical personnel at the time of construction.

If the granular pavement materials are to be used by construction traffic, it may be necessary to increase the thickness of the granular subbase layer.The contractor should be made responsible for their construction access.

4.6.3 Asphalt Cement Type

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

4.6.4 Pavement Transitions

As part of the access roadway/parking lot construction, the new pavement will abut the existing pavement at Teron Road and various locations. The following is suggested to improve the performance of the joint between the new and the existing pavements:

- Neatly saw cut the existing asphaltic concrete;
- Remove the asphaltic concrete and slope the bottom of the excavation within the existing granular base and subbase at 1 horizontal to 1 vertical, or flatter, to avoid undermining the existing asphaltic concrete.
- To avoid cracking of the asphaltic concrete due to an abrupt change in the thickness of the roadway granular materials where new pavement areas join with the existing pavements, the granular depths should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the existing pavement structure.
- Remove (mill off) 40 to 50 millimetres of the existing asphaltic concrete to a distance of 300 millimetres at the joint and tack coat the asphaltic concrete at the joint in accordance with the requirements in OPSS 310.

4.6.5 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 proposed parking areas grades are generally lower than the existing grades at the site and, due to the peizometric pressure in the bedrock at this site, we recommend that a permeable drainage layer be incorporated in the pavement structure in order to intercept the upward flow of groundwater from the bedrock. We recommended that the drainage layer be composed of a full length plastic perforated subdrains spaced at about 5 metres on centre within the proposed parking lot at subgrade level. The drains should outlet by gravity to catch basins or ditches.

Catch basins should be equipped with minimum 3 metre long stub drains extending in two directions at the subgrade level.

4.6.6 Granular Material Compaction

The granular base and subbase materials should be compacted in maximum 300 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density value.

4.7 Post-Construction (Long-Term) Groundwater Pumping

For a finished floor elevation of 85.5 metres, the floor slab, within the southeast portion of the building footprint, will be located below the measured groundwater levels. Further, the proposed parking lot grades within the southeast portion of the site are below the measured groundwater levels. Assuming a hydraulic conductivity of 1 x 10^{-5} metres per second for bedrock, the amount of groundwater collected within the drainage layer installed below the slab and parking area could be in the order of 400,000 litres per day, based on the groundwater levels measured on March 14, 2020.

It should be noted that the flow into bedrock excavations is expected to vary dependent upon the presence/absence of discrete water bearing fractures, as evident by the range of calculated bedrock hydraulic conductivities. Furthermore, no information is available on the long-term groundwater levels throughout the year; however, it is expected that the rate of groundwater pumping will be less during drier periods of the year.

The groundwater quantity provided above was calculated based on a large diameter well approximation, and on a linear flow model. The estimated quantity does not take into account groundwater infiltration during periods of precipitation and snow melt.

We recommend that the groundwater quantities be measured during construction to check the above estimates.

5.0 ADDITIONAL CONSIDERATIONS

5.1 Effects of Construction Induced Vibration

Some of the construction operations (such as excavation, hoe ramming, granular material compaction, etc.) will cause ground vibration on and off of 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. We recommend that preconstruction surveys be carried out on the adjacent structures and that vibration monitoring be carried out during bedrock excavation to ensure that vibrations are below typical threshold values and so that any damage claims can be addressed in a fair manner.

5.2 Corrosion of Buried Concrete and Steel

The measured sulphate concentration from the samples of soil recovered from 19-3, 19-6, 19-7 and 19-10 range from <5 to 100 micrograms per gram. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate can be classified as low. Therefore, any concrete in contact with the native soil could be batched with General Use (GU) cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) use on the roadway should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

The measured sulphate concentration from the groundwater sample recovered from 20-2 was 17 milligrams per litre. According to Canadian Standards Association (CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate can be classified as low. Therefore, any concrete in contact with the groundwater could be batched with General Use (GU) cement. The effects of freeze thaw in the presence of de-icing chemical (sodium chloride) use onsite should be considered in selecting the air entrainment and the concrete mix proportions for any concrete.

Based on the resistivity and pH of the samples, the soil and groundwater in this area can be classified as non-aggressive towards unprotected steel. It should be noted that the corrosivity of the soil/groundwater could vary throughout the year due to the application sodium chloride for de-icing.

5.3 Winter Construction

The soils that exist at this site are highly frost susceptible and are prone to significant ice lensing. In the event that construction is required during freezing temperatures, the soil below the footings and floor slabs should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any service trenches should be opened for as short a time as practicable and the excavations should be carried out only in lengths that allow all of the construction operations, including backfilling, to be fully completed in one working day. The materials on the sides of the trenches should not be allowed to freeze. In addition, the backfill should be excavated, stored and replaced without being disturbed by frost or contaminated by snow or ice.

5.4 Excess Soil Management Plan

This report does not constitute an excess soil management plan. The disposal requirements for excess soil from the site have not been assessed.

5.5 Landscape Design

The City of Ottawa document titled: "Tree Planting in Sensitive Marine Soils - 2017 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.

For this site, low/medium potential clay soils encompass the entire property.

In accordance with the City of Ottawa Tree Planting Guidelines, tree setback 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 finished grade (refer to the City of Ottawa document titled: "Tree Planting in Sensitive Marine Soils - 2017 Guidelines"). In areas where clay soils are not present within 3.5 metres of finished grade (e.g., where bedrock is at founding level or where relatively thick pads of engineered fill are required below founding level), the City of Ottawa tree planting restrictions may not apply. Given the considerable grading (i.e. cut/fill) work to be completed on this site, it is recommended that the grades be reviewed by GEMTEC prior to the completion of the landscape plan so that tree planting restrictions can be identified.

5.6 Design Review

The design details of the proposed development were not available to us at the time of preparation of this report. It is recommended that the design drawings be reviewed by the geotechnical engineer as the design progresses to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the proposed development should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

6.0 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.

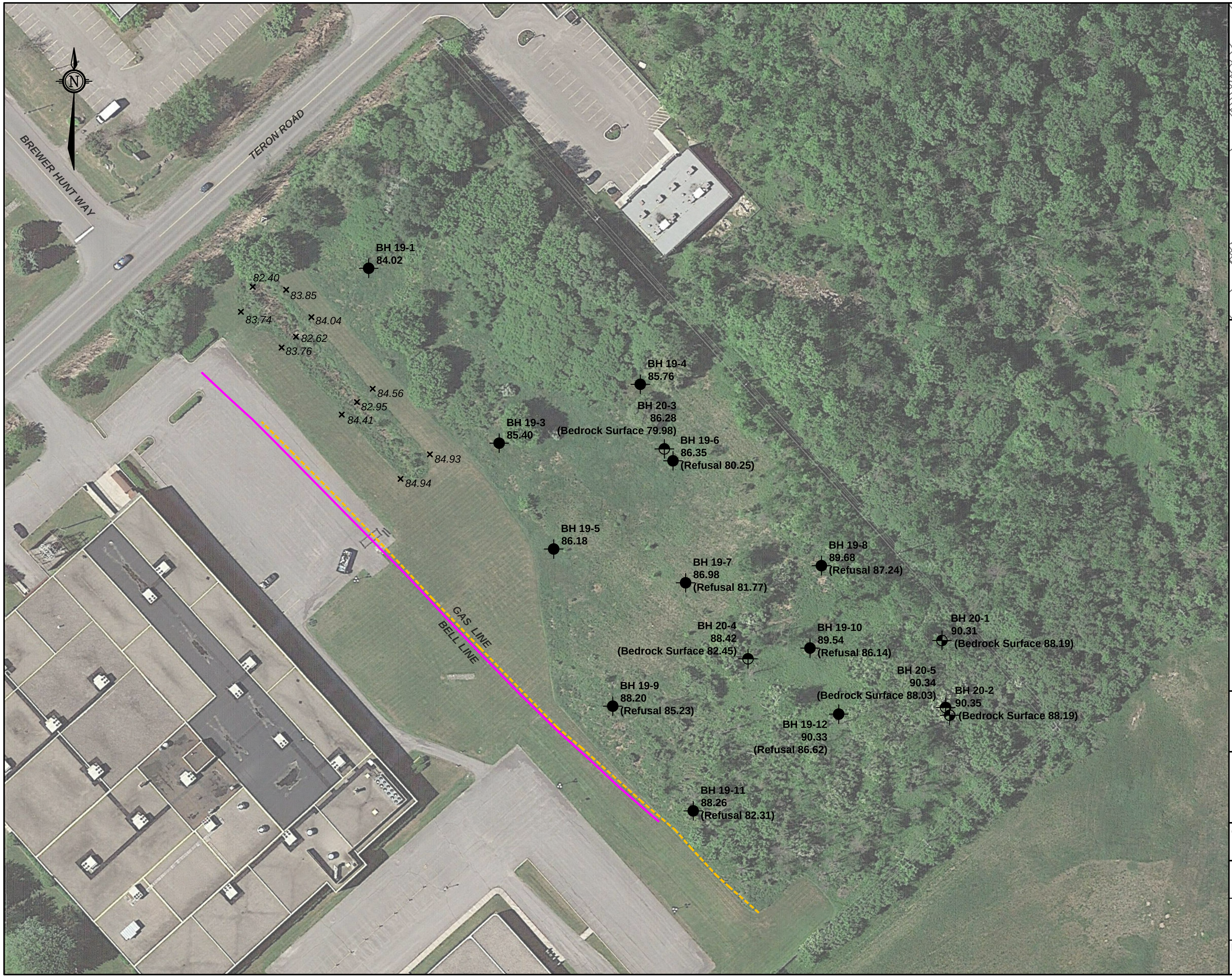
G. Davidson

Gregory Davidson, P.Eng.
Geotechnical Engineer

Johnathan A. Cholewa, Ph.D., P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J. A. CHOLEWA
100119055
30 Oct 2020
PROVINCE OF ONTARIO

GD/JC



KEY PLAN
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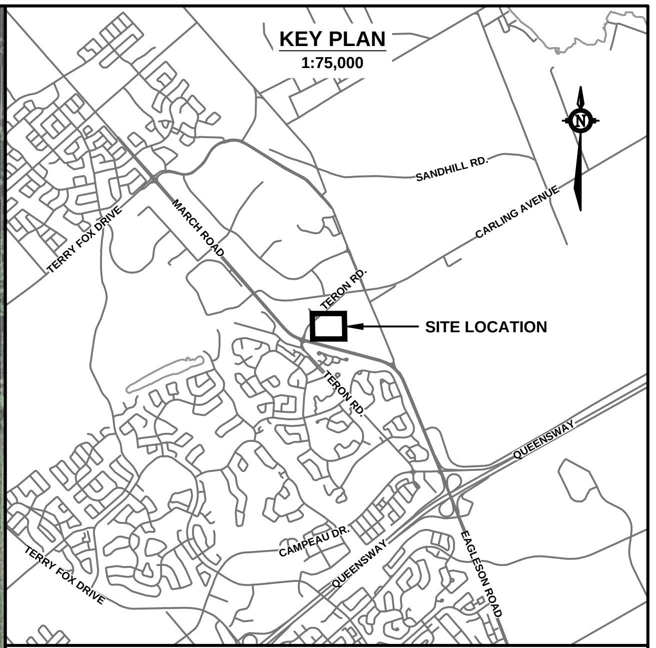

LEGEND

⊕ **BOREHOLE LOCATION IN PLAN**
(investigation by GEMTEC, June 2020)

◐ **BOREHOLE LOCATION IN PLAN**
(investigation by GEMTEC, March 2020)

● **BOREHOLE LOCATION IN PLAN**
(investigation by GEMTEC, August 2019)

BH # ← BOREHOLE ID
XX.XX ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

× *84.93* ← GROUND SURFACE ELEVATION, IN METRES GEODETIC DATUM

NOTE:

LOCATION OF UTILITIES ON THIS PLAN ARE APPROXIMATE AND ARE FOR REFERENCE ONLY.

Scale 1:1000
0 20 40 60m

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

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www.gemtec.ca
ottawa@gemtec.ca

Client MEGHA HOLDINGS INC.	Project 64742.02
Location 1243 TERON ROAD, OTTAWA, ON	

Drwn by P.C.	Chkd by L.B.	BOREHOLE LOCATION PLAN
Date JULY 2020	Rev. 2	FIGURE 1

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APPENDIX A

List of Abbreviations and Terminology
Record of Borehole Sheets

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

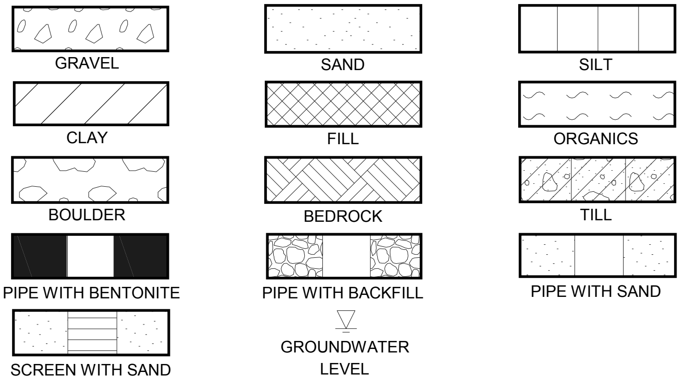
PENETRATION RESISTANCE

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.

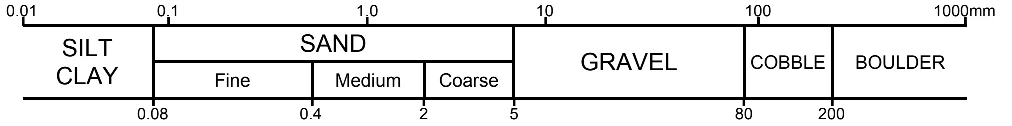
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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.

GEMTEC

descriptive terms.pub
Modified May 2018

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE	
Fresh	No visible sign of rock material weathering
Faintly weathered	Weathering limited to the surface of major discontinuities
Slightly weathered	Penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material
Moderately weathered	Weathering extends throughout the rock mass but the rock material is not friable
Completely weathered	Rock is wholly decomposed and in a friable condition but the rock and structure are preserved

CORE CONDITION
Total Core Recovery (TCR) The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run
Solid Core Recovery (SCR) The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.
Rock Quality Designation (RQD) The percentage of solid drill core, greater than 100 mm length, as measured along the centerline axis of the core, relative to the length of the total core run. RQD varies from 0% for completed broken core to 100% for core in solid segments.

BEDDING THICKNESS	
Description	**Thickness**
Thinly laminated	< 6 mm
Laminated	6 - 20 mm
Very thinly bedded	20 - 60 mm
Thinly bedded	60 - 200 mm
Medium bedded	200 - 600 mm
Thickly bedded	600 - 2000 mm
Very thickly bedded	2000 - 6000 mm

DISCONTINUITY SPACING	
Description	**Spacing**
Very close	20 - 60 mm
Close	60 - 200 mm
Moderate	200 - 600 mm
Wide	600 -2000 mm
Very wide	2000 - 6000 mm

ROCK QUALITY	
RQD	**Overall Quality**
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

ROCK COMPRESSIVE STRENGTH	
Comp. Strength, MPa	**Description**
1 - 5	Very weak
5 - 25	Weak
25 - 50	Moderate
50 - 100	Strong
100 - 250	Very strong

RECORD OF BOREHOLE 19-1

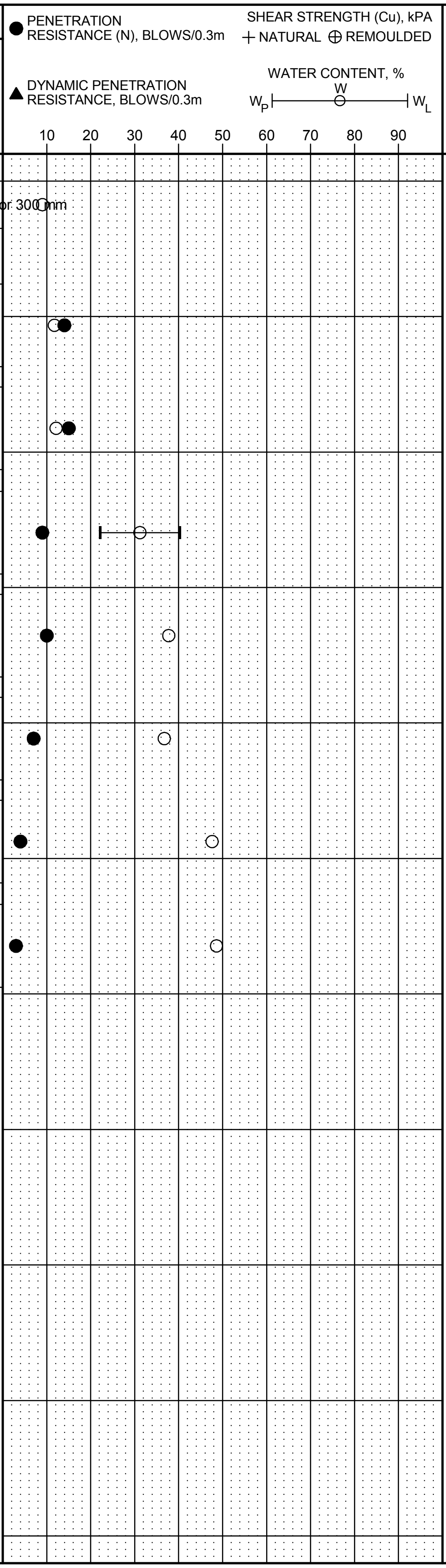
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 15 2019

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)
0	Ground Surface		84.02
	Dark brown silty sand with organic material (TOPSOIL)		0.10
	Compact, brown silty sand, trace to some clay and gravel (FILL MATERIAL)		
2	Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)		81.89 2.13
6	End of Borehole		78.07 5.95

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	75	>50 for 300 mm
2	SS	125	14
3	SS	300	15
4	SS	425	9
5	SS	600	10
6	SS	600	7
7	SS	600	4
8	SS	600	3



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with soil cuttings

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Consulting Engineers and Scientists

LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 19-3

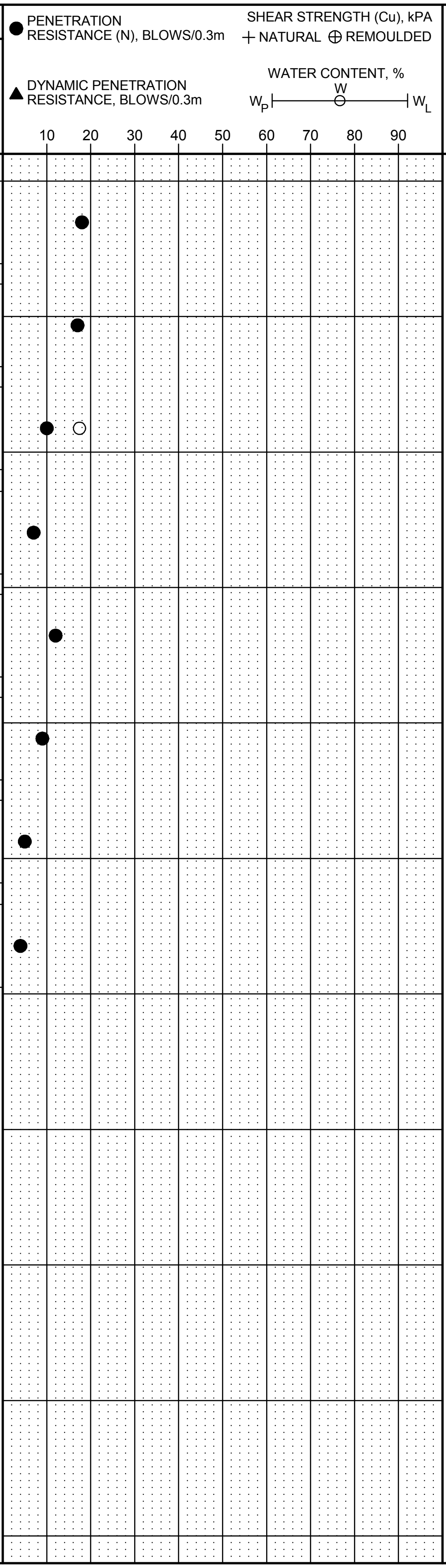
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 15 2019

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	85.40
	Dark brown silty sand with organic material (TOPSOIL)	0.10
	Compact, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	
2	Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	83.27 2.13
6	End of Borehole	79.45 5.95

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	100	18	
2	SS	425	17	
3	SS	300	10	MH
4	SS	425	7	
5	SS	600	12	
6	SS	600	9	
7	SS	600	5	
8	SS	600	4	



PIEZOMETER OR STANDPIPE INSTALLATION

Backfilled with soil cuttings

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LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 19-4

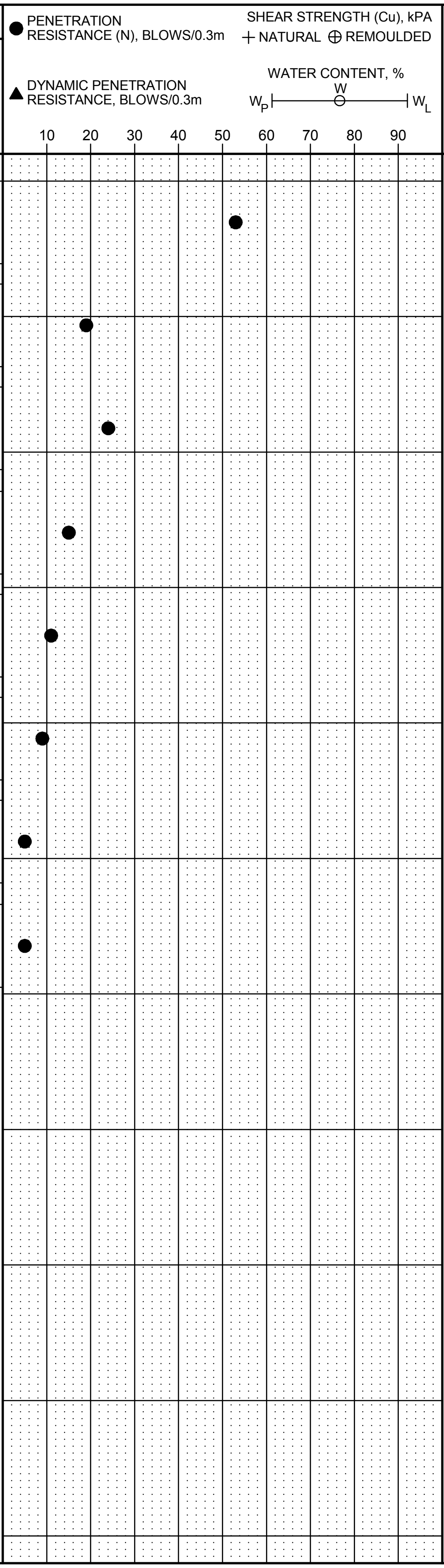
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	85.76
	Dark brown silty sand with organic material (TOPSOIL)	0.10
	Compact to dense, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	
2	Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	83.63 2.13
6	End of Borehole	79.81 5.95

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	50	53
2	SS	250	19
3	SS	350	24
4	SS	500	15
5	SS	600	11
6	SS	600	9
7	SS	600	5
8	SS	600	5



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with soil cuttings

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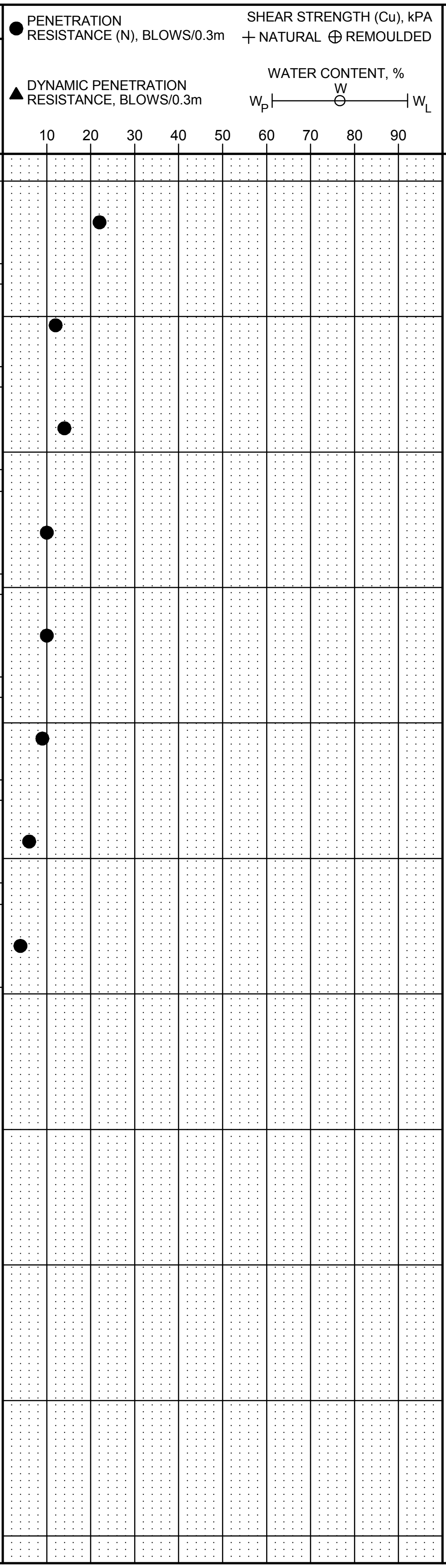
CHECKED: G.D.

RECORD OF BOREHOLE 19-5

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 15 2019

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger Hollow Stem Auger (210mm OD)	Ground Surface	86.18						
		Dark brown silty sand with organic material (TOPSOIL)							Backfilled with soil cuttings
		Compact, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	0.10	1	SS	200	22		
1				2	SS	300	12		
		Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	84.66 1.52	3	SS	600	14		
2				4	SS	600	10		
3				5	SS	600	10		
4				6	SS	600	9		
5				7	SS	600	6		
				8	SS	600	4		
6		End of Borehole	80.23 5.95						
7									
8									
9									
10									



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LOGGED: M.L.

CHECKED: G.D.

RECORD OF BOREHOLE 19-6

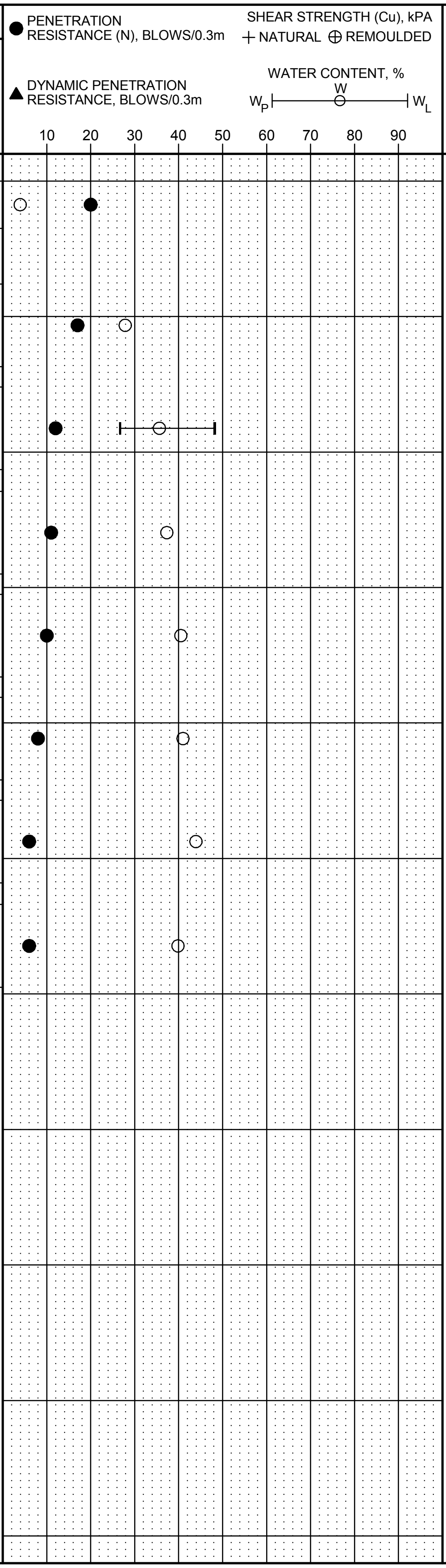
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

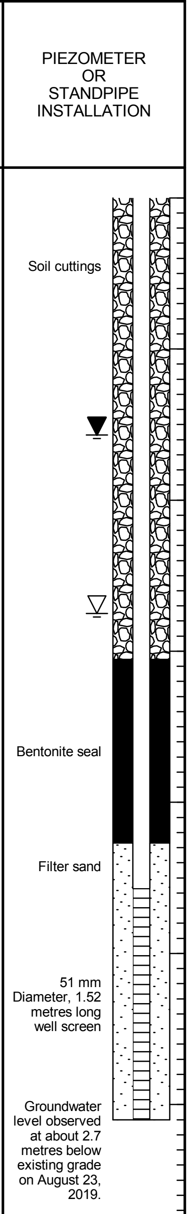
SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	86.35
Dark brown silty sand with organic material (TOPSOIL)	0.08
Brown silty sand, trace to some clay and gravel (FILL MATERIAL)	
Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	85.59 0.76
Brown silt and sand, some gravel (GLACIAL TILL)	80.56 5.79
End of Borehole Auger Refusal	80.25 6.10

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	325	20	
2	SS	200	17	
3	SS	425	12	wp
4	SS	600	11	
5	SS	600	10	
6	SS	600	8	
7	SS	600	6	
8	SS	600	6	





GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)		ELEV. (m)
19/08/23	2.8	▽	83.6
20/06/24	1.6	▼	84.8

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LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 19-7

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

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

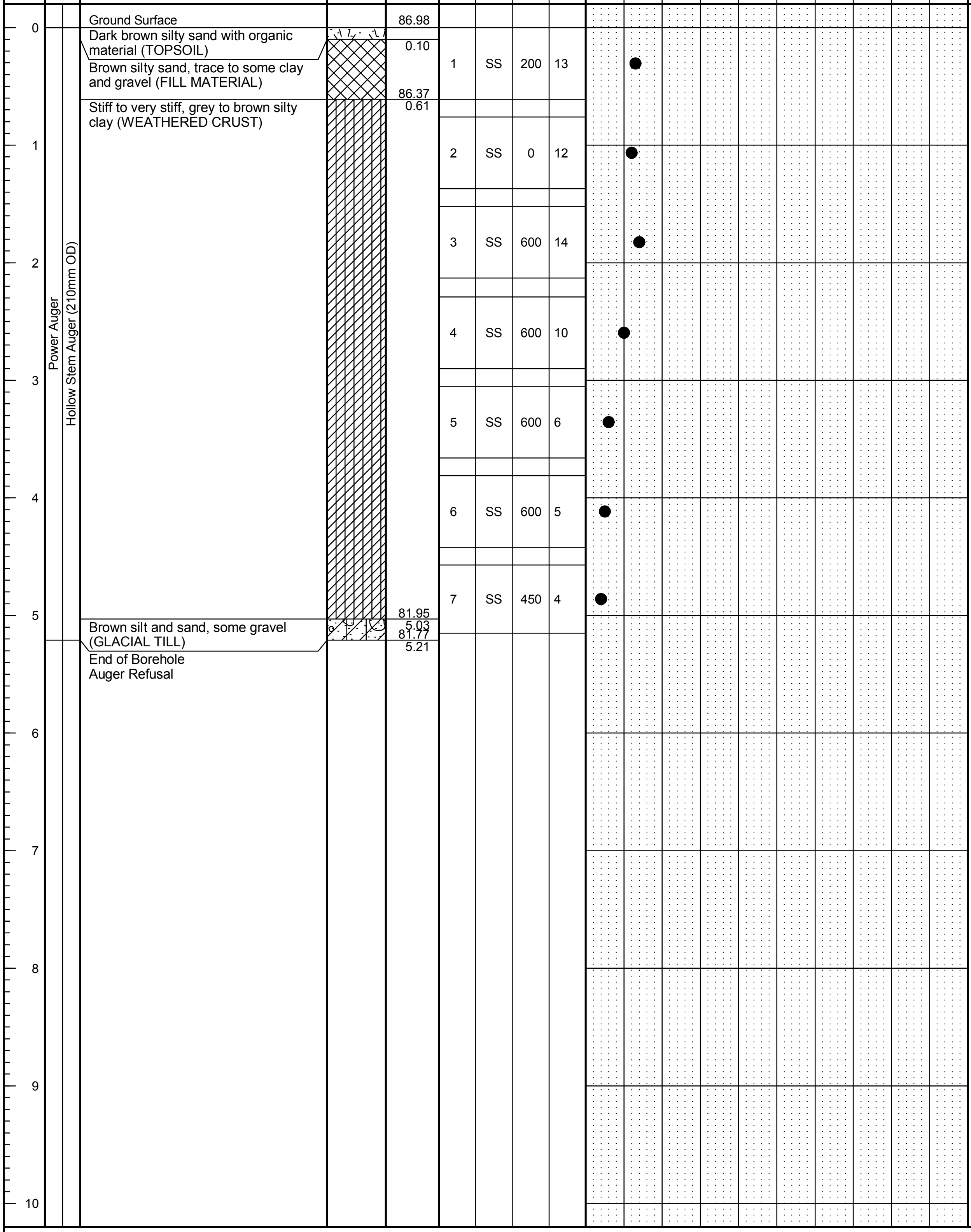
SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	86.98
Dark brown silty sand with organic material (TOPSOIL)	0.10
Brown silty sand, trace to some clay and gravel (FILL MATERIAL)	86.37 0.61
Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	81.95 5.03
Brown silt and sand, some gravel (GLACIAL TILL)	81.77 5.21
End of Borehole Auger Refusal	

SAMPLES

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	200	13
2	SS	0	12
3	SS	600	14
4	SS	600	10
5	SS	600	6
6	SS	600	5
7	SS	450	4

● 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

Backfilled with soil cuttings

LOGGED: M.L.
CHECKED: G.D.

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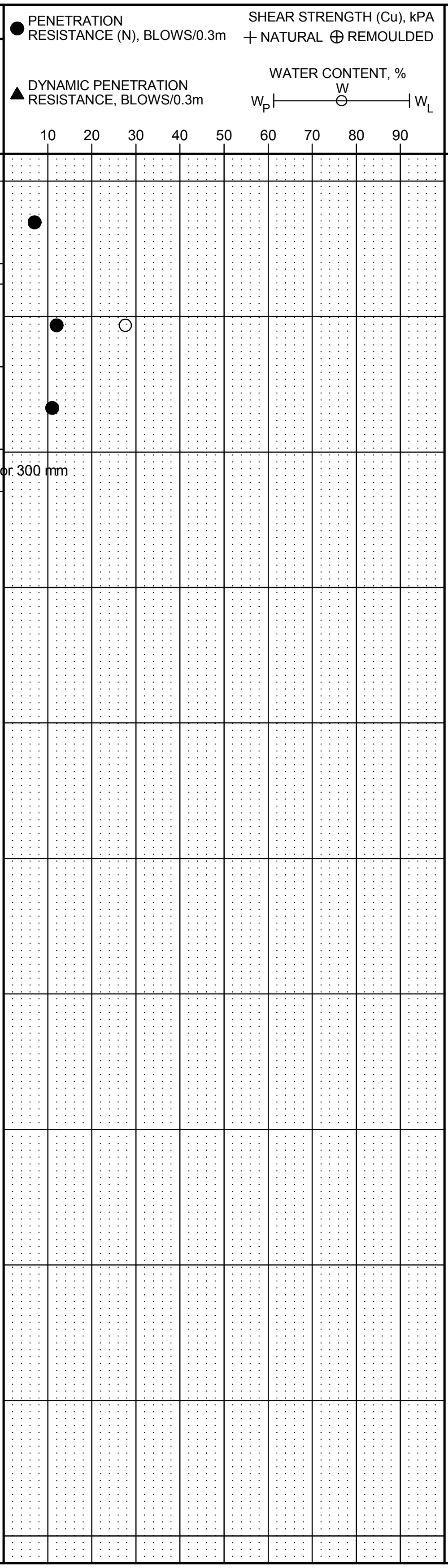
RECORD OF BOREHOLE 19-8

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger Hollow Stem Auger (210mm OD)	Ground Surface	89.68						
		Dark brown silty sand with organic material (TOPSOIL)	0.05						Backfilled with soil cuttings
		Loose, brown silty sand, trace to some clay and gravel (FILL MATERIAL)		1	SS	200	7		
		Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	89.07 0.61						
1				2	SS	300	12	MH	
				3	SS	450	11		
2				4	SS	200	>50 for 300 mm		
		End of Borehole Auger Refusal	87.24 2.44						
3									
4									
5									
6									
7									
8									
9									
10									



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LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 19-9

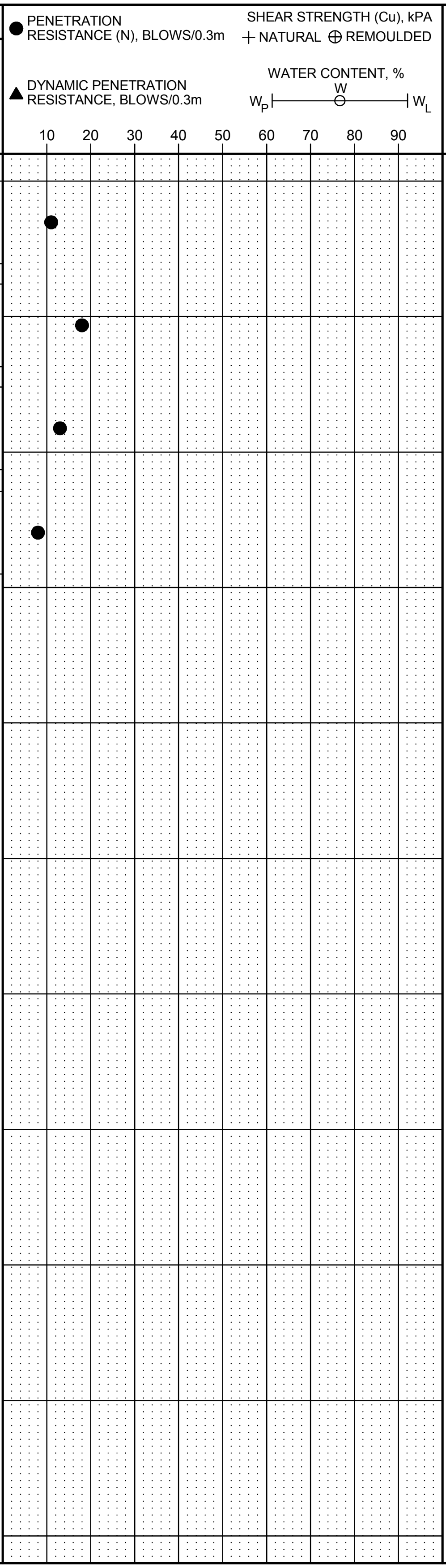
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	88.20
	Dark brown silty sand with organic material (TOPSOIL)	
	Compact, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	0.10
1	Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	87.13 1.07
	Brown silt and sand, some gravel (GLACIAL TILL)	85.46 2.74
3	End of Borehole Auger Refusal	85.23 2.97

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	200	11
2	SS	300	18
3	SS	400	13
4	SS	100	8



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with soil cuttings (0 to 3 m)

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LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 19-10

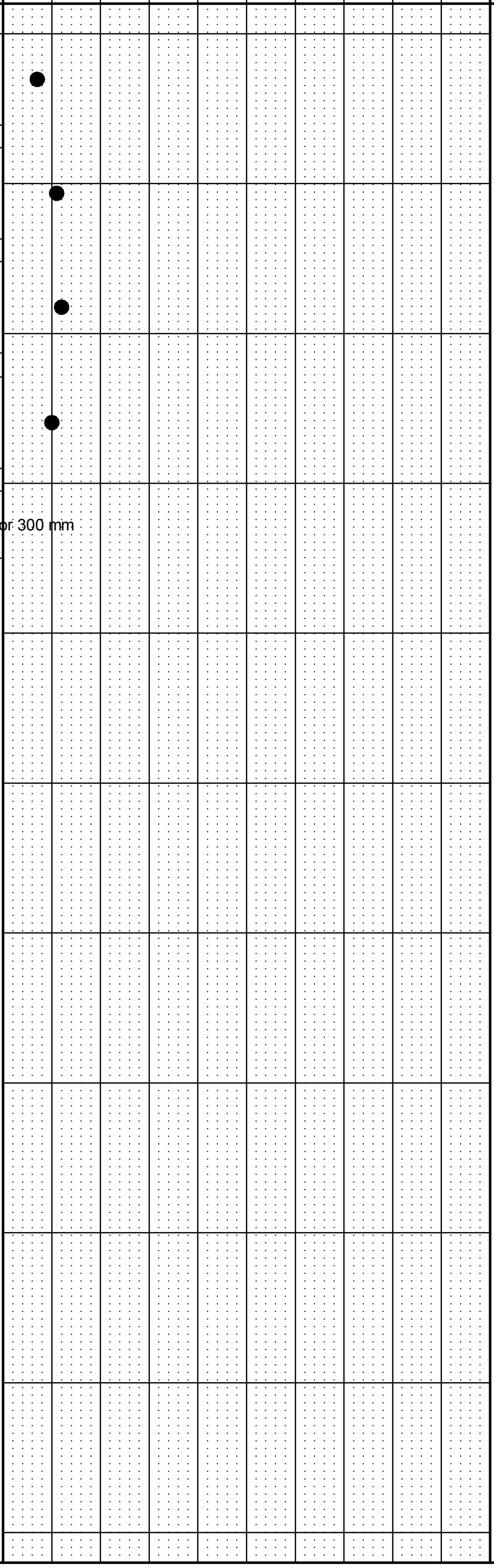
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

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

SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
Ground Surface	89.54
Dark brown silty sand with organic material (TOPSOIL)	0.05
loose, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	
Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	88.71 0.83
Brown silt and sand, some gravel (GLACIAL TILL)	86.23 3.31
End of Borehole Auger Refusal	3.40

SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	250	7
2	SS	275	11
3	SS	450	12
4	SS	600	10
5	SS	450	>50 for 300 mm



ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION: Backfilled with soil cuttings

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Consulting Engineers and Scientists

LOGGED: M.L.
CHECKED: G.D.

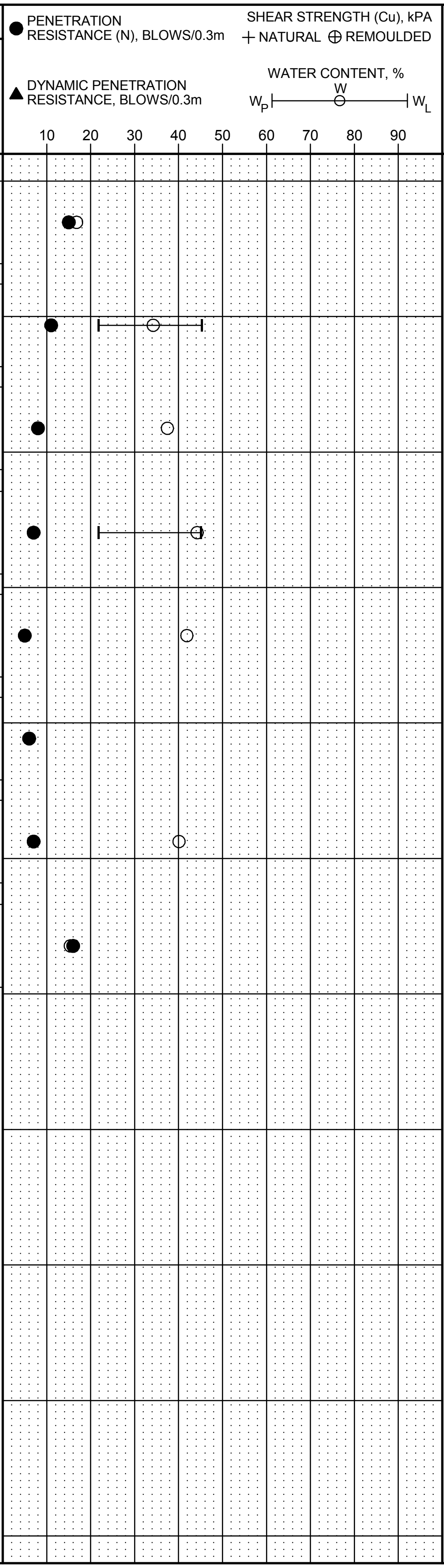
RECORD OF BOREHOLE 19-11

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 15 2019

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
Power Auger Hollow Stem Auger (210mm OD)	Ground Surface	88.26						
	Dark brown silty sand with organic material (TOPSOIL)	0.10						
	Compact, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	87.65 0.61	1	SS	100	15		Backfilled with soil cuttings
	Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)		2	SS	600	11		
			3	SS	600	8		
			4	SS	600	7		
			5	SS	600	5		
			6	SS	600	6	MH	
		82.92 5.34	7	SS	600	7		
	Compact, grey silt and sand, some gravel (GLACIAL TILL)	82.31 5.95	8	SS	600	16	M	
	End of Borehole Auger Refusal							

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



GEMTEC
Consulting Engineers and Scientists

LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 19-12

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Aug 16 2019

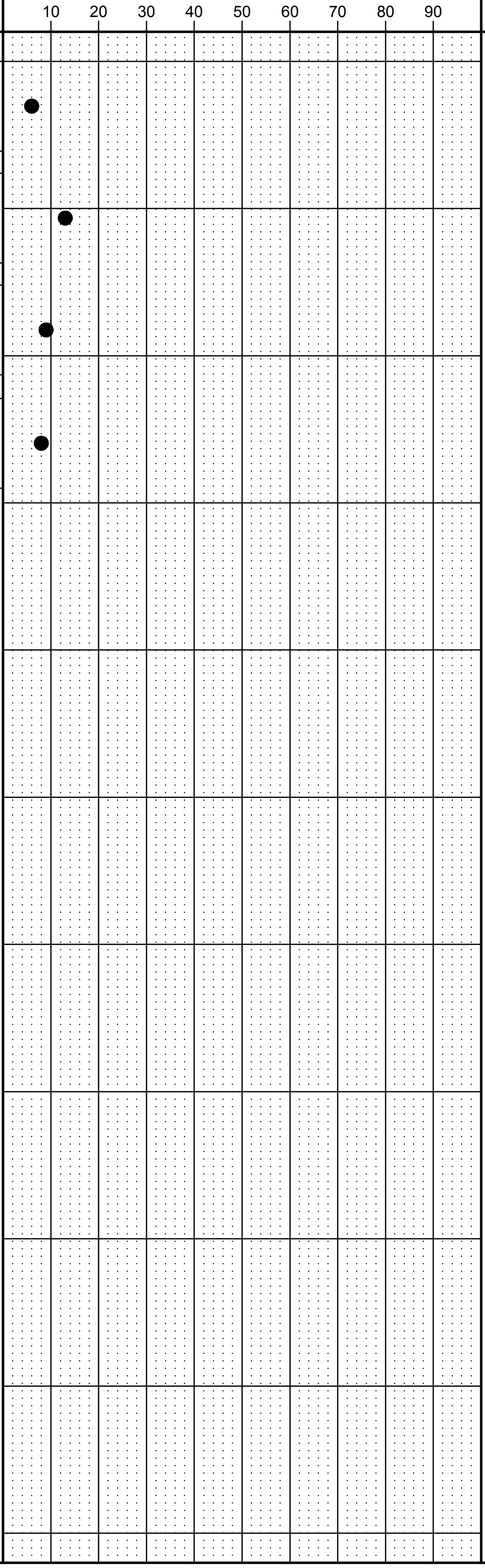
BORING METHOD: Power Auger, Hollow Stem Auger (210mm OD)

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)
0	Ground Surface	89.67
	Dark brown silty sand with organic material (TOPSOIL)	0.05
	Compact, brown silty sand, trace to some clay and gravel (FILL MATERIAL)	
	Stiff to very stiff, grey to brown silty clay (WEATHERED CRUST)	88.91 0.76
3	End of Borehole Auger Refusal	86.62 3.05

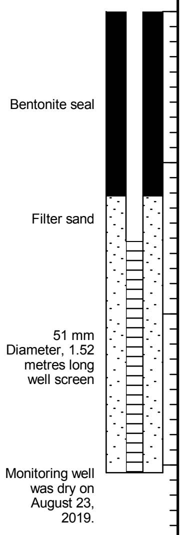
NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
1	SS	300	6
2	SS	350	13
3	SS	600	9
4	SS	600	8

● 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



Monitoring well was dry on August 23, 2019.

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LOGGED: M.L.
CHECKED: G.D.

RECORD OF BOREHOLE 20-1

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Mar 4 2020

● 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
10 20 30 40 50 60 70 80 90

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m		ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		90.32							
0–2	Power Auger Hollow Stem Auger (210mm OD)	Overburden not sampled, refer to previous investigation for approximate soil statigraphy									Bentonite seal
2	Diamond Rotary Core HQ (89mm OD)	Grey and pink, slightly weathered, moderately fracture GRANITIC BEDROCK, some oxidation at joints	▨	88.19 2.13	1	RC			TCR = 100%, SCR = 55%, RQD = 55%		
3			▨		2	RC			TCR = 100%, SCR = 66%, RQD = 56%		
4		End of Borehole		86.41 3.91							
5											
6											
7											
8											
9											
10											

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LOGGED: ML
CHECKED: LB

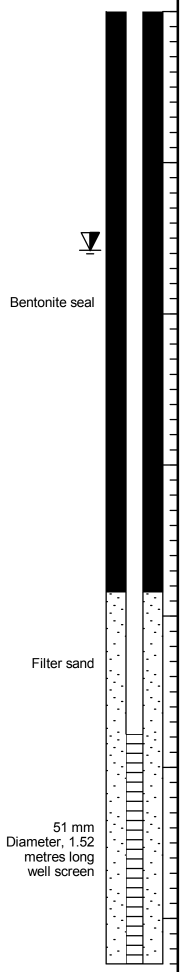
RECORD OF BOREHOLE 20-2

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation - 1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Mar 3 2020

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	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) 10 20 30 40 50 60 70 80 90	ADDITIONAL LAB. TESTING
0		Ground Surface		90.35						
0 – 2.16	Power Auger Hollow Stem Auger (210mm OD)	Overburden not sampled, refer to previous investigation for approximate soil statigraphy								
2.16 – 6.30	Diamond Rotary Core HQ (89mm OD)	Grey and pink, slightly weathered, moderately fracture GRANITIC BEDROCK, some oxidation at joints	▨	88.19 2.16	1	RC			TCR = 81%, SCR = 37%, RQD = 30%	
					2	RC			TCR = 100%, SCR = 60%, RQD = 49%	
					3	RC			TCR = 100%, SCR = 48%, RQD = 27%	
					4	RC			TCR = 100%, SCR = 100%, RQD = 89%	
6.30		End of Borehole		84.05 6.30						

PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)		ELEV. (m)
20/03/10	-0.3	▽	90.7
20/03/14	-0.4	▼	90.7
20/06/24	1.6	▼	88.8

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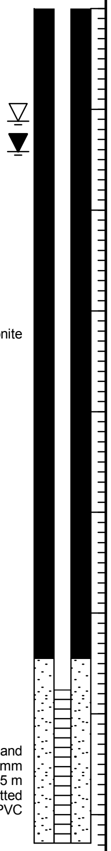
LOGGED: ML
CHECKED: LB

RECORD OF BOREHOLE 20-03

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation -1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 10 2020

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	● 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 (10 20 30 40 50 60 70 80 90)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0		Ground Surface		86.28							
	Wash Casing HW (114mm OD)	Subsurface conditions not logged									Bentonite
6	Diamond Rotary Core HQ (89mm OD)	Grey and pink, slightly weathered, close to moderatley fractured GRANITE BEDROCK -Some oxidation at joints	▨	79.98 6.30	1	RC		TCR = 97%; SCR = 67%; RQD= 50%			Filter Sand 50 mm diameter, 1.5 m length slotted SCH 40 PVC
7					2	RC		TCR = 100%; SCR = 81%; RQDD = 72%			
8		End of borehole		78.00 8.28							



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)		ELEV. (m)
20/06/17	1.1	▽	85.2
20/06/24	1.4	▼	84.9

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Consulting Engineers and Scientists

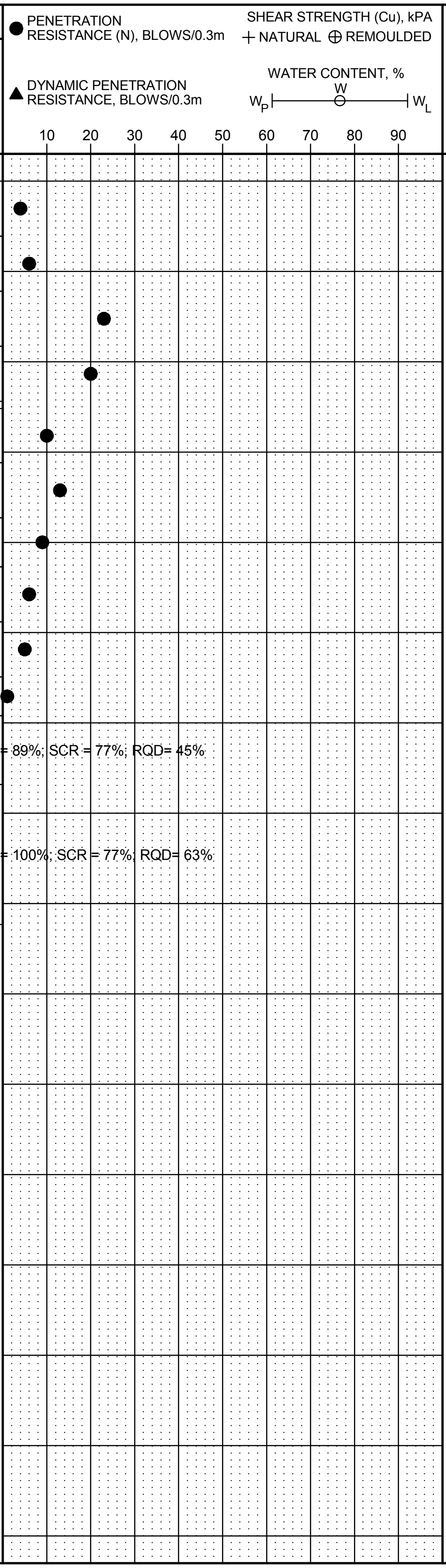
LOGGED: A.N.
CHECKED: G.D.

RECORD OF BOREHOLE 20-04

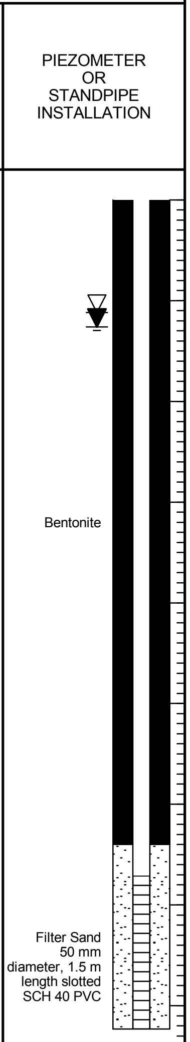
CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation -1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 10 2020

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
	Ground Surface	88.42				
Power Auger Hollow Stem Auger (210mm OD)	Dark brown silty sand with organic material (TOPSOIL)	88.22 0.20	1	SS	432	4
	Brown silty sand (FILL MATERIAL)	87.86 0.56				
	Stiff to very stiff, grey brown SILTY CLAY (WEATHERED CRUST)		2	SS	457	6
			3	SS	610	23
			4	SS	203	20
			5	SS	610	10
			6	SS	610	13
			7	SS	610	9
			8	SS	203	6
			9	SS	610	5
		82.45 5.97	10	SS	483	1
Diamond Rotary Core HQ (89mm OD)	Grey and pink, slightly weathered, close to moderatley fractured GRANITE BEDROCK -Some oxidation at joints		11	RC		TCR = 89%; SCR = 77%; RQD= 45%
		80.19 8.23	12	RC		TCR = 100%; SCR = 77%; RQD= 63%
	End of borehole					



ADDITIONAL LAB. TESTING



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)		ELEV. (m)
20/06/17	1.1	▽	87.3
20/06/24	1.3	▼	87.2

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Consulting Engineers and Scientists

LOGGED: A.N.

CHECKED: G.D.

RECORD OF BOREHOLE 20-05

CLIENT: Megha Holdings Inc.
PROJECT: Geotechnical Investigation -1243 Teron Road
JOB#: 64742.02
LOCATION: See Borehole Location Plan, Figure 1

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Jun 9 2020

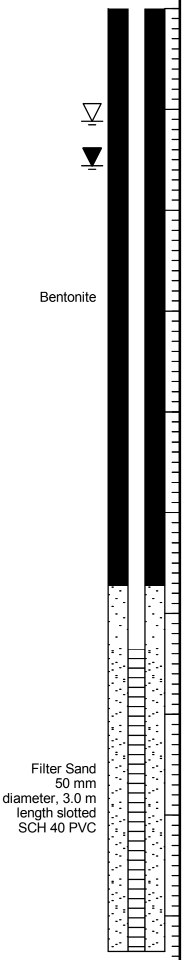
● PENETRATION RESISTANCE (N), BLOWS/0.3m
▲ DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m
SHEAR STRENGTH (Cu), kPA: + NATURAL ⊕ REMOULDED
WATER CONTENT, %: W_P ⊢ W ⊣ W_L
10 20 30 40 50 60 70 80 90

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)
0		Ground Surface	90.34
0–2.31	Wash Casing HW (114mm OD)	Subsurface conditions not logged	
2.31–9.35	Diamond Rotary Core HQ (89mm OD)	Grey and pink, slightly weathered, close to moderatley fractured GRANITE BEDROCK -Some oxidation at joints	88.03 2.31
9.35		End of borehole	80.99 9.35

NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	
1	RC			TCR = 100%; SCR = 72%; RQD= 20%
2	RC			TCR = 100%; SCR = 65%; RQD= 40%
3	RC			TCR = 100%; SCR = 67%; RQD= 35%
4	RC			TCR = 100%; SCR = 67%; RQD= 35%
5	RC			TCR = 100%; SCR = 55%; RQD= 48%

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
20/06/17	1.3 ▽	89.0
20/06/24	1.6 ▼	88.8

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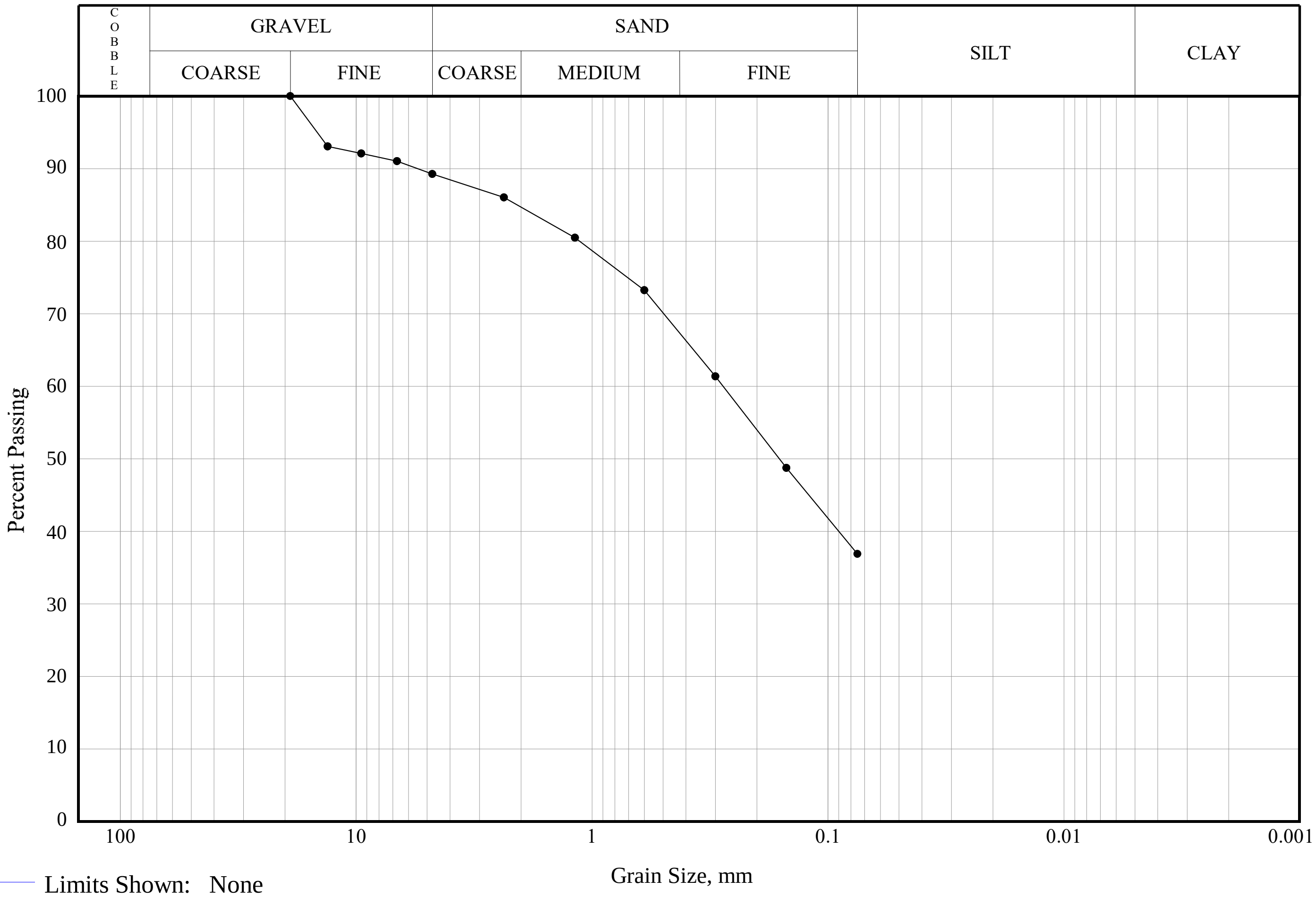
LOGGED: A.N.
CHECKED: G.D.

APPENDIX B

Laboratory Testing Results
Soils Grading Chart
Plasticity Chart

GEMTEC Consulting Engineers and Scientists		
Client:	Megha Holdings Inc.	
Project:	1243 Teron Road, Ottawa	
Project #:	6474202	

Soils Grading Chart



Limits Shown: None

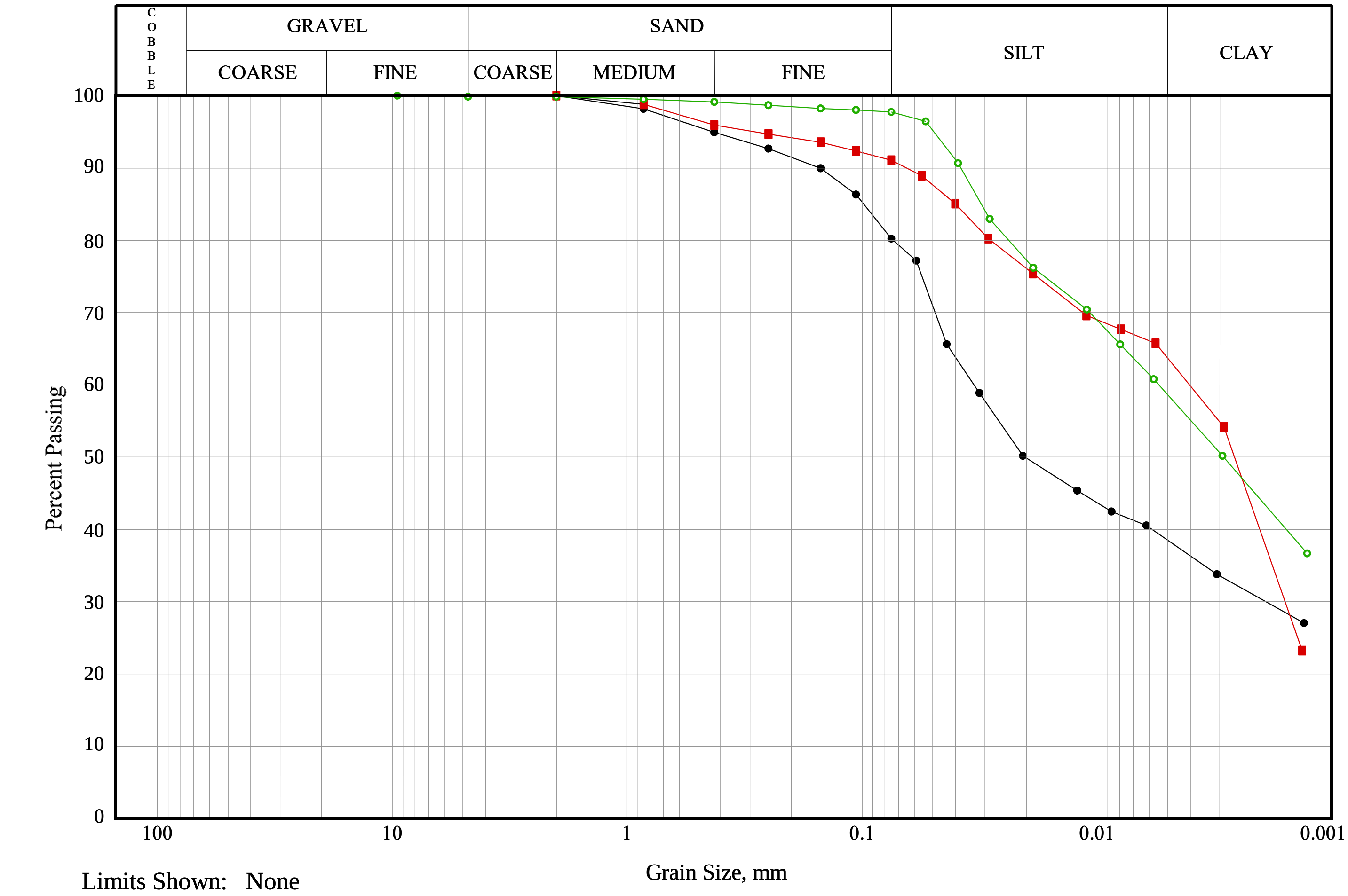
Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Glacial Till	19-11	08	5.33-5.94	10.8	52.4	36.9	

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75μm
—●—	Sand and silt , some gravel	N/A	---	---	---	0.16	0.28	2.08	---

GEMTEC
Consulting Engineers
and Scientists

Client: Megha Holdings Inc.
Project: 1243 Teron Road, Ottawa
Project #: 64742.02

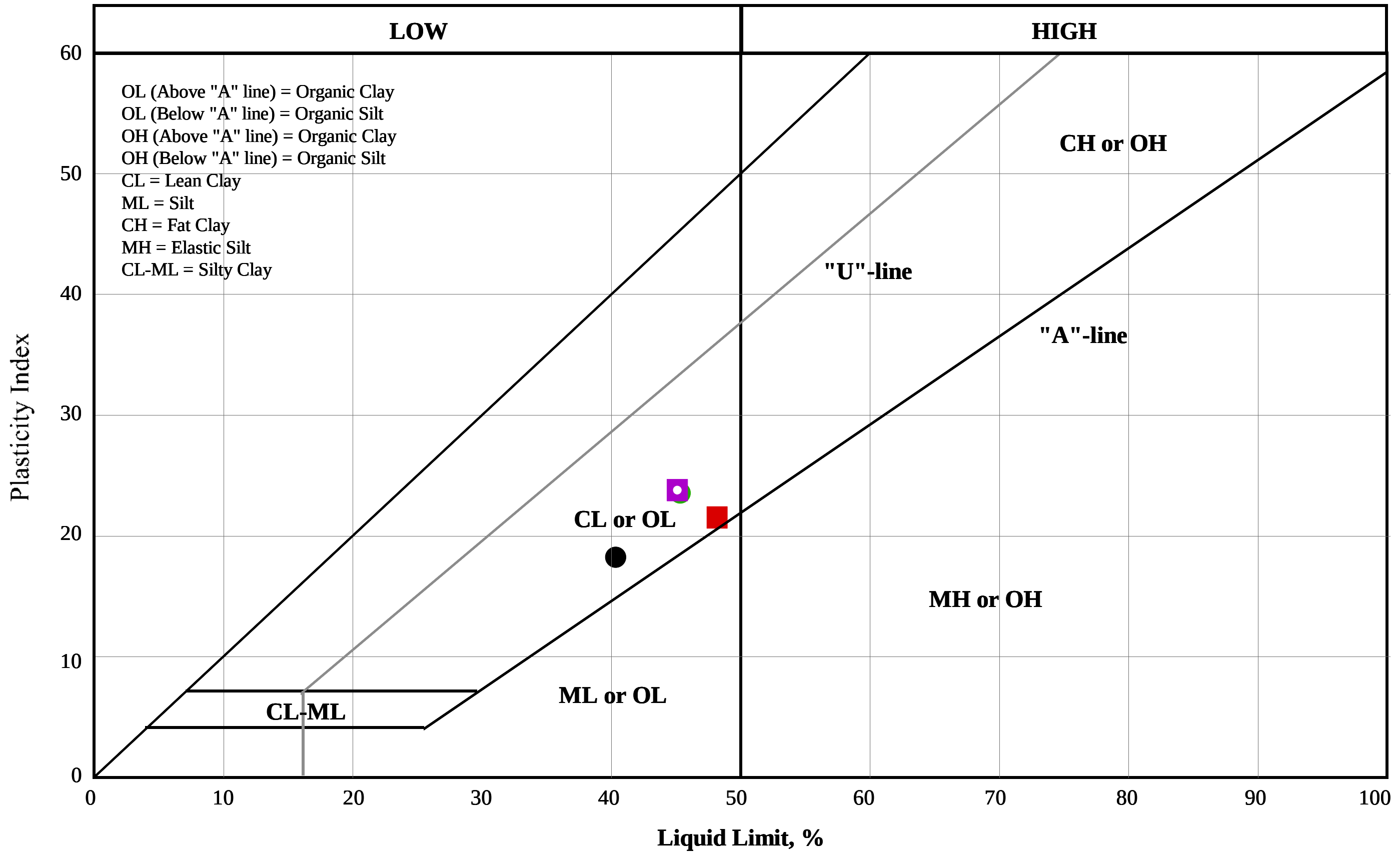
Soils Grading Chart



Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
●	Fill Material	19-3	03	1.52-2.13	0.0	19.8	41.7	38.5
■	Weathered Crust	19-8	02	0.76-1.37	0.0	8.9	27.4	63.7
○	Weathered Crust	19-11	06	3.81-4.42	0.1	2.1	39.1	58.6

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75µm
●	Silt and clay , some sand	N/A	---	---	0.00	0.02	0.03	0.10	41.7
■	Silty clay , trace sand	N/A	---	---	0.00	0.00	0.00	0.04	27.4
○	Clay and silt , trace gravel, trace sand	N/A	---	---	---	0.00	0.01	0.03	39.1

GEMTEC Consulting Engineers and Scientists	Client: Megha Holdings Inc. Project: 1243 Teron Road, Ottawa Project #: 6474202	# Plasticity Chart



Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	19-1	04	2.29-2.90	40.3	22.1	18.2	☐	31.19
■	19-6	03	1.52-2.13	48.2	26.7	21.5	☐	35.62
○	19-11	02	0.76-1.37	45.3	21.8	23.5	☐	34.22
▣	19-11	04	2.29-2.90	45.1	21.3	23.8	☐	44.23

APPENDIX C

Chemical Analysis of Soil and Groundwater Relating to Corrosion (Paracel Laboratories Ltd. Order No. 1935274 and 2011212)

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 30-Aug-2019

Order Date: 27-Aug-2019

Project Description: 64742.02

	Client ID: **Sample Date:** **Sample ID:**	19-3 SA4 16-Aug-19 09:00 1935274-01	19-6 SA3 16-Aug-19 09:00 1935274-02	19-7 SA3 16-Aug-19 09:00 1935274-03	19-10 SA2 16-Aug-19 09:00 1935274-04
	MDL/Units	Soil	Soil	Soil	Soil
Physical Characteristics					
% Solids	0.1 % by Wt.	78.8	75.1	75.3	75.9
General Inorganics					
Conductivity	5 uS/cm	175	57	70	38
pH	0.05 pH Units	7.82	8.13	7.71	7.43
Resistivity	0.10 Ohm.m	57.1	175	142	265
Anions					
Chloride	5 ug/g dry	7	7	8	6
Sulphate	5 ug/g dry	100	9	30	<5

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 13-Mar-2020

Order Date: 10-Mar-2020

Project Description: 64742.02

	Client ID:	BH20-2 GW-1	-	-	-
	Sample Date:	10-Mar-20 13:00	-	-	-
	Sample ID:	2011212-01	-	-	-
	MDL/Units	Water	-	-	-
General Inorganics					
pH	0.1 pH Units	7.8	-	-	-
Resistivity	0.01 Ohm.m	32.1	-	-	-
Anions					
Chloride	1 mg/L	5	-	-	-
Sulphate	1 mg/L	17	-	-	-

APPENDIX D

Bedrock Core Photos
Unconfined Compressive Strength

BOREHOLE 20-1 - RC1 + RC2

GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS
32 Steacie Drive, Ottawa, ON K2K 2A9
T: (613) 836-1422 | www.gemtec.ca | ottawa@gemtec.ca

Project

PROPOSED COMMERCIAL BUILDING
1243 TERON ROAD

FIGURE D1

File No.
64742.02

BEDROCK CORE PICTURES - BOREHOLE 20-1

BOREHOLE 20-2 - RC1 + RC2

BOREHOLE 20-2 - RC3 + RC4

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BEDROCK CORE PICTURES - BOREHOLE 20-2

Project	Project No.	
PROPOSED COMMERCIAL BUILDING 1243 TERON ROAD	64742.02	**FIGURE D2**

BOREHOLE 20-3 - RC1 + RC2

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Project
PROPOSED COMMERCIAL BUILDING
1243 TERON ROAD

FIGURE D3

File No.
64742.02

BEDROCK CORE PICTURES - BOREHOLE 20-3

BOREHOLE 20-4 - RC1 + RC2

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Project PROPOSED COMMERCIAL BUILDING 1243 TERON ROAD	**FIGURE D4**		BEDROCK CORE PICTURES - BOREHOLE 20-4
	File No. 64742.02		

BOREHOLE 20-5 - RC1 + RC2 + RC3 + RC4 + RC5

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Project

PROPOSED COMMERCIAL BUILDING
1243 TERON ROAD

FIGURE D5

File No.
64742.02

BEDROCK CORE PICTURES - BOREHOLE 20-5

GEMTEC Consulting Engineers and
Scientists Limited
32 Steacie Drive
Ottawa, ON
K2K 2A9
Tel.: 613-836-1422
Fax.:613-836-9731

COMPRESSIVE STRENGTH
of BEDROCK CORE

CLIENT: Megha Holdings Inc.

PROJECT No.: 64742.02

Project:

REPORT NO:

Date Received: 6-Mar-20

Date Tested: 10-Mar-20

Lab no.	RC-1	RC-4				
Cylinder ID	BH20-1	BH20-2				
Depth (m)	2.4-2.6	6.0-6.2				
Cut length (mm)	83.47	124.96				
Ground length (mm)	81.32	123.28				
Diameter (mm)	63.31	63.20				
Ground Mass (kg)	0.65	1.02				
Length:Diameter ratio	1.28	1.95				
Correction factor	0.93	1.00				
Failure load (kN)	124.69	287.75				
Uncorrected Strength (MPa)	39.60	91.70				
Corrected Strength (MPa)	**36.80**	**91.70**				

Remarks

Checked by: ______________________
Krystle Smith, Laboratory Manager

Reviewed by: ______________________
Steve Goodman, Ph.D., P.Eng.

GEMTEC Consulting Engineers and
Scientists Limited
32 Steacie Drive
Ottawa, ON
K2K 2A9
Tel.: 613-836-1422
Fax.:613-836-9731

COMPRESSIVE STRENGTH of ROCK CORE

CLIENT: Megha Holdings Inc.

PROJECT No.: 64742.02

Project: 1243 Teron Road

REPORT NO: 1

Date Received: 16-Jun-20

Date Tested: 16-Jun-20

Lab no.	-	-				
Cylinder ID	20-3 RC1	20-5 RC2				
Depth (m)	6.47-6.73	8.25-8.50				
Cut length (mm)	-	-				
Ground length (mm)	123.60	120.77				
Diameter (mm)	63.25	63.11				
Ground Mass (kg)	106.00	107.00				
Length:Diameter ratio	1.95	1.91				
Correction factor	1.00	0.99				
Failure load (kN)	624.59	158.87				
Uncorrected Strength (MPa)	198.80	50.80				
Corrected Strength (MPa)	**198.80**	**50.30**				

Remarks

Checked by: Krystle Smith, Laboratory Manager

Reviewed by: Steve Goodman, Ph.D., P.Eng.

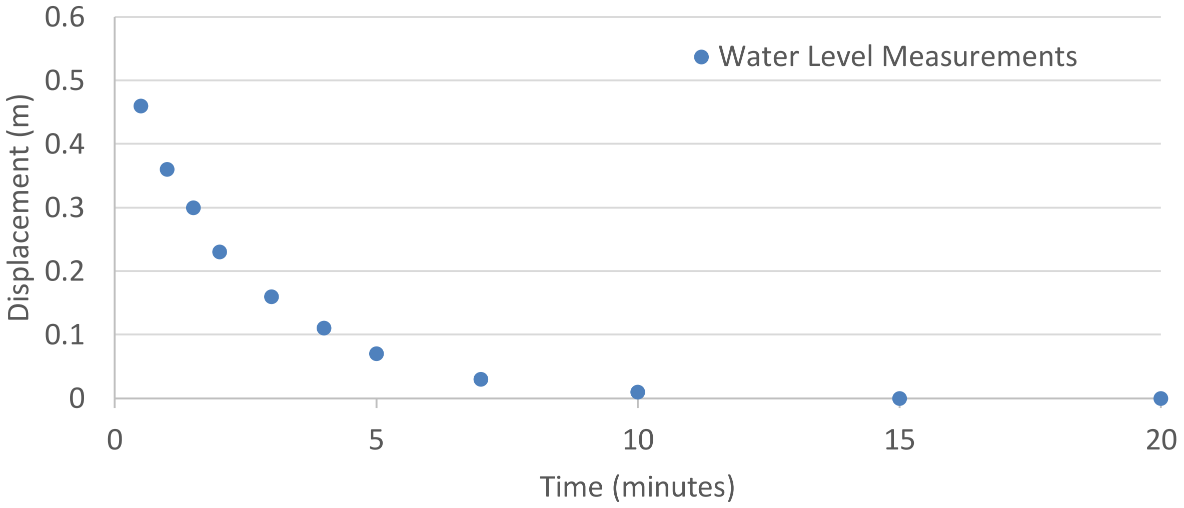
APPENDIX E

Hydraulic Testing Results
Figures E1 to E7

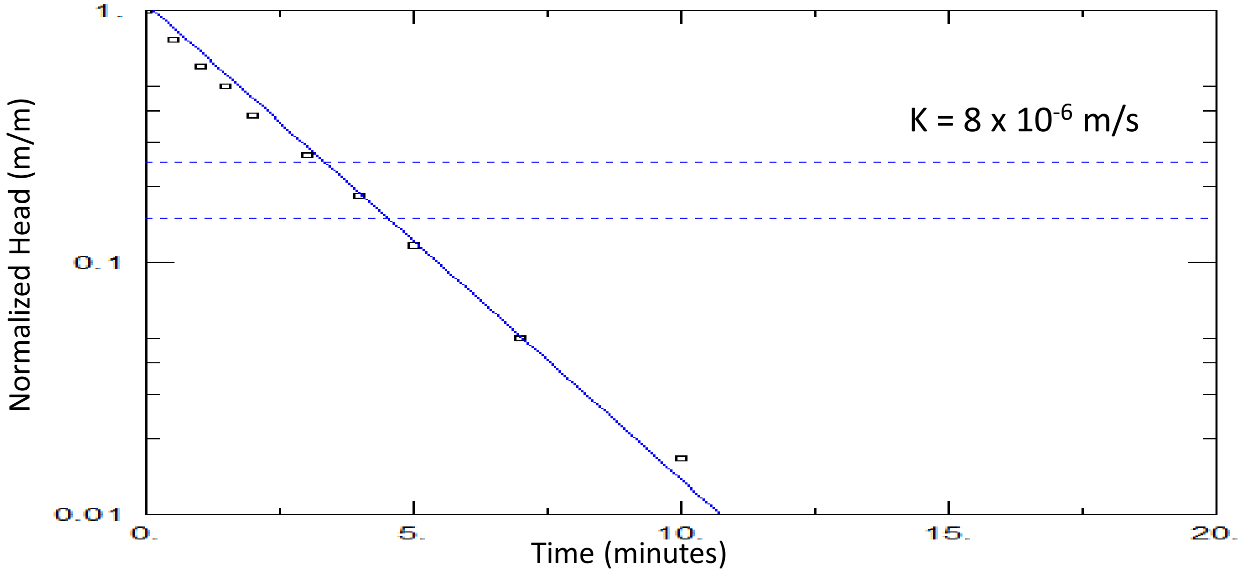
Hydraulic Testing

FIGURE E1

Borehole 20-2 Rising Head (RH) Test



Borehole 20-2 RH Aqtesolv Analysis



Well Data:

Displacement observed (slug size): 0.56 metres (0.60 m)
Well Depth: 6.30 metres
Screen Length: 1.52 metres
Well Radius: 0.085 metres

Aquifer Data

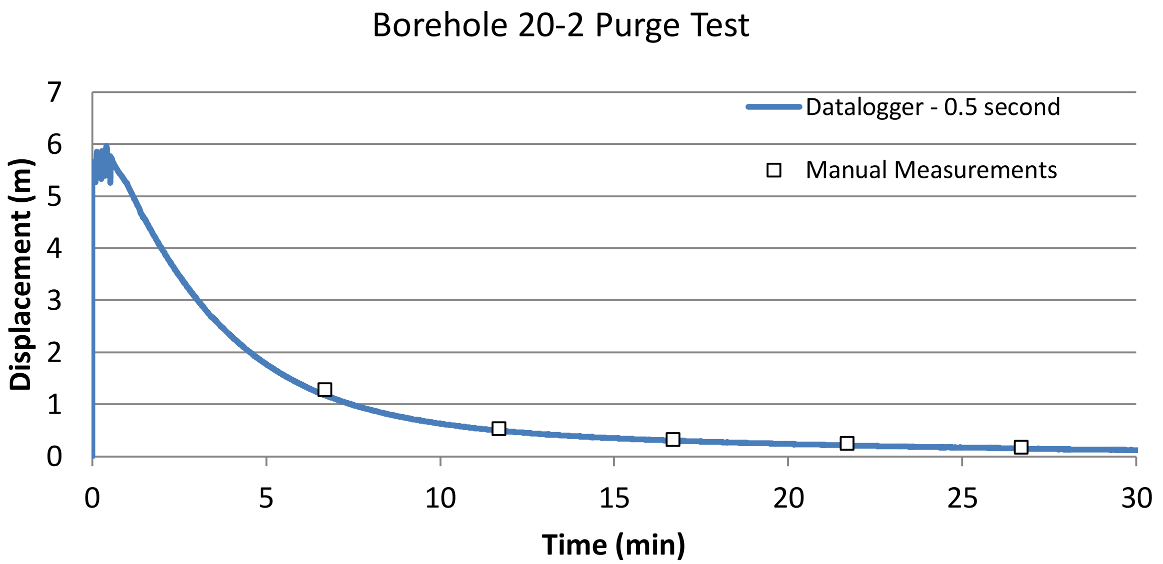
Saturated Thickness: 1.52 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined, Hvorslev
Static Water Level: -0.4 metres bgs

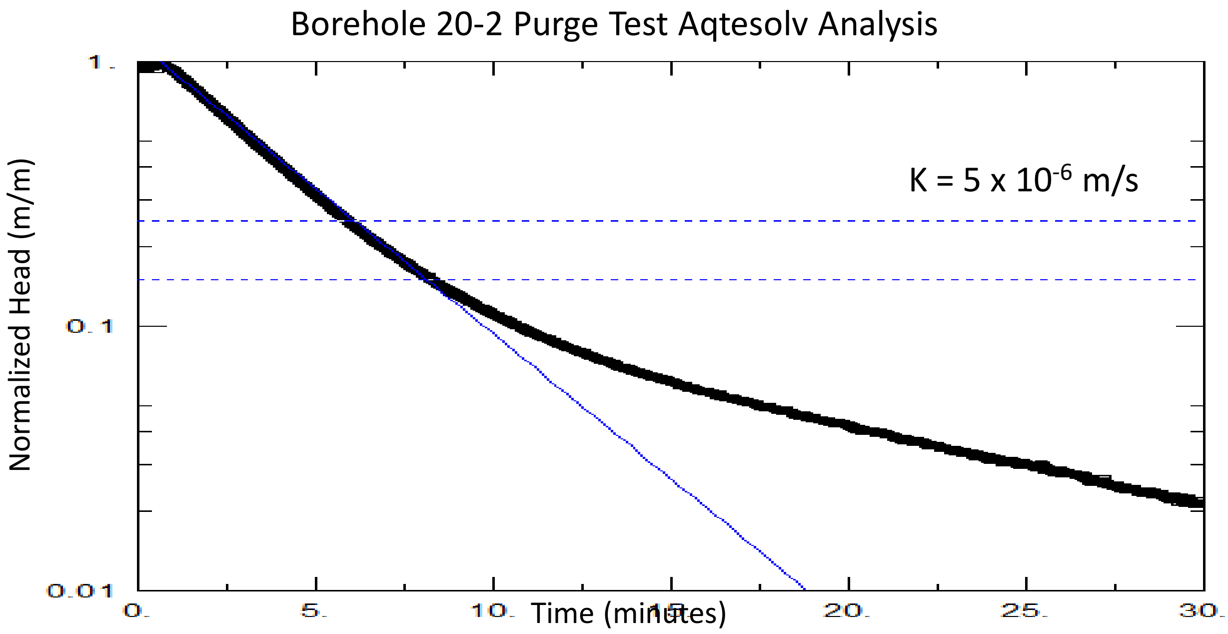
GEMTEC
CONSULTING ENGINEERS AND SCIENTISTS

Date: March 2020
Project: 64742.02

Hydraulic Testing

FIGURE E2





<u>Well Data:</u>

Displacement observed following purging: 5.66 m
Well Depth: 6.30 metres
Screen Length: 1.52 metres
Well Radius: 0.085 metres

<u>Aquifer Data</u>

Saturated Thickness: 1.52 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined, Hvorslev
Static Water Level: -0.4 metres bgs

GEMTEC
CONSULTING ENGINEERS
AND SCIENTISTS

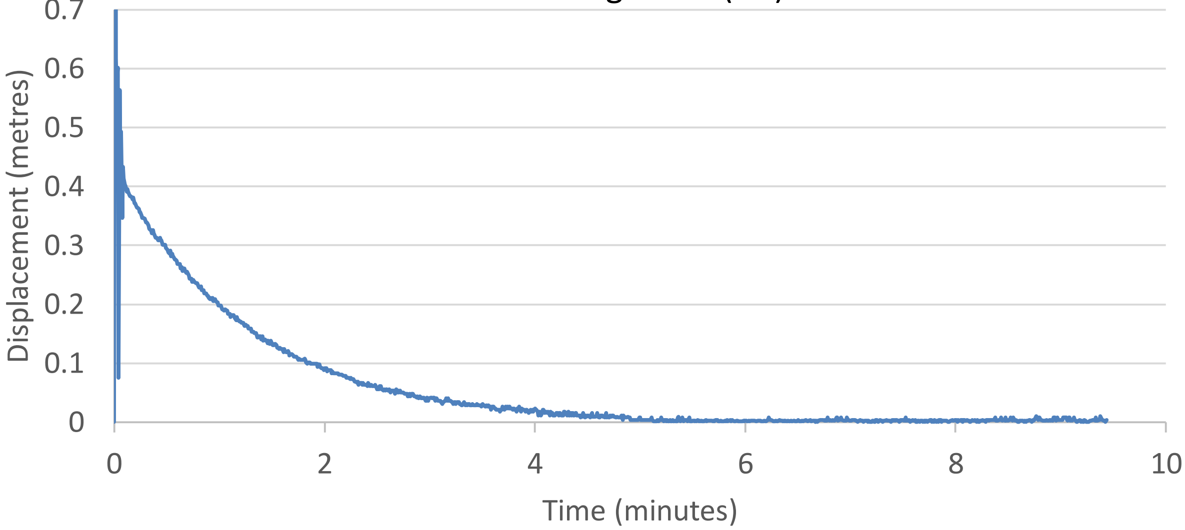
Date: March 2020

Project: 64742.02

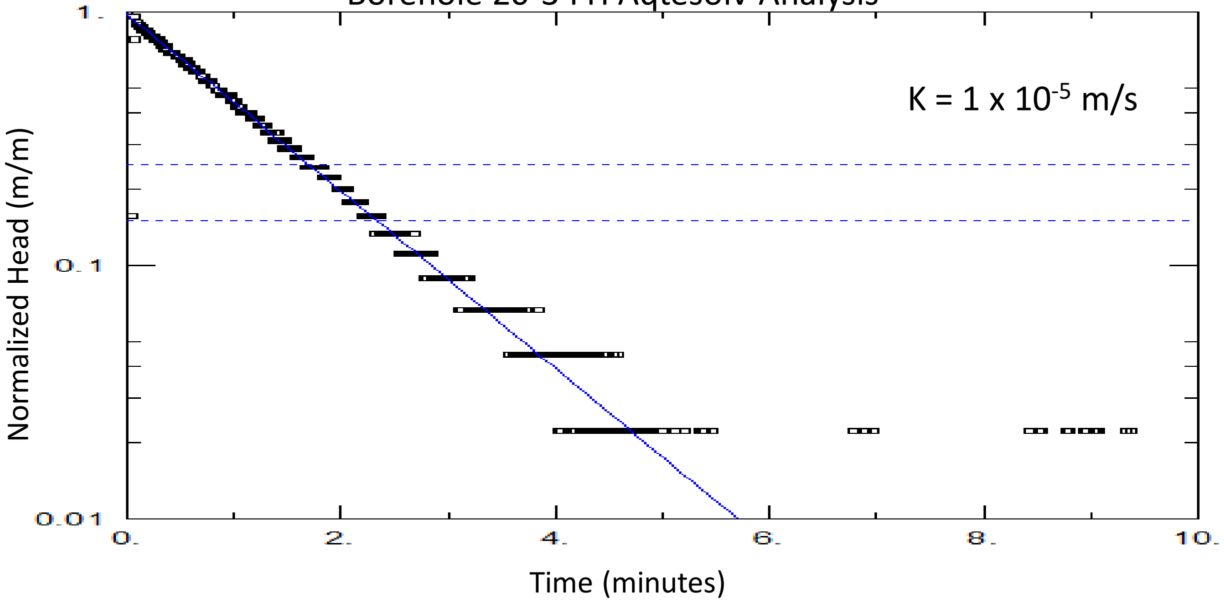
Hydraulic Testing

FIGURE E3

Borehole 20-3 Falling Head (FH) Test



Borehole 20-3 FH Aqtesolv Analysis



Well Data:
Displacement observed (slug size): 2.01 metres (0.45 m)
Well Depth: 8.28 metres
Screen Length: 1.52 metres
Well Radius: 0.085 metres

Aquifer Data
Saturated Thickness: 1.52 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined, Hvorslev
Static Water Level: 1.12 metres bgs

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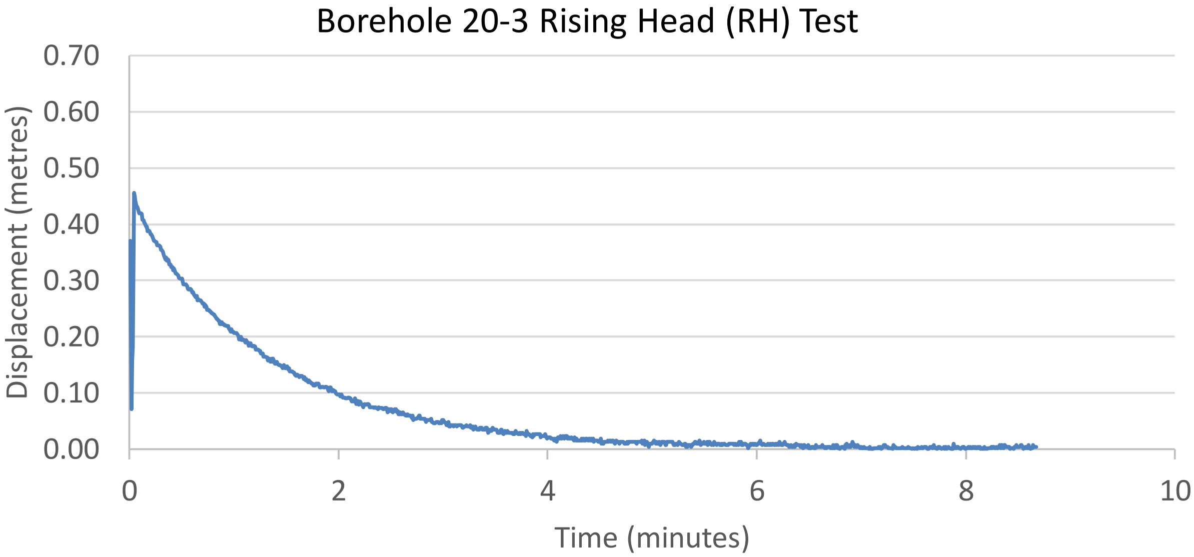
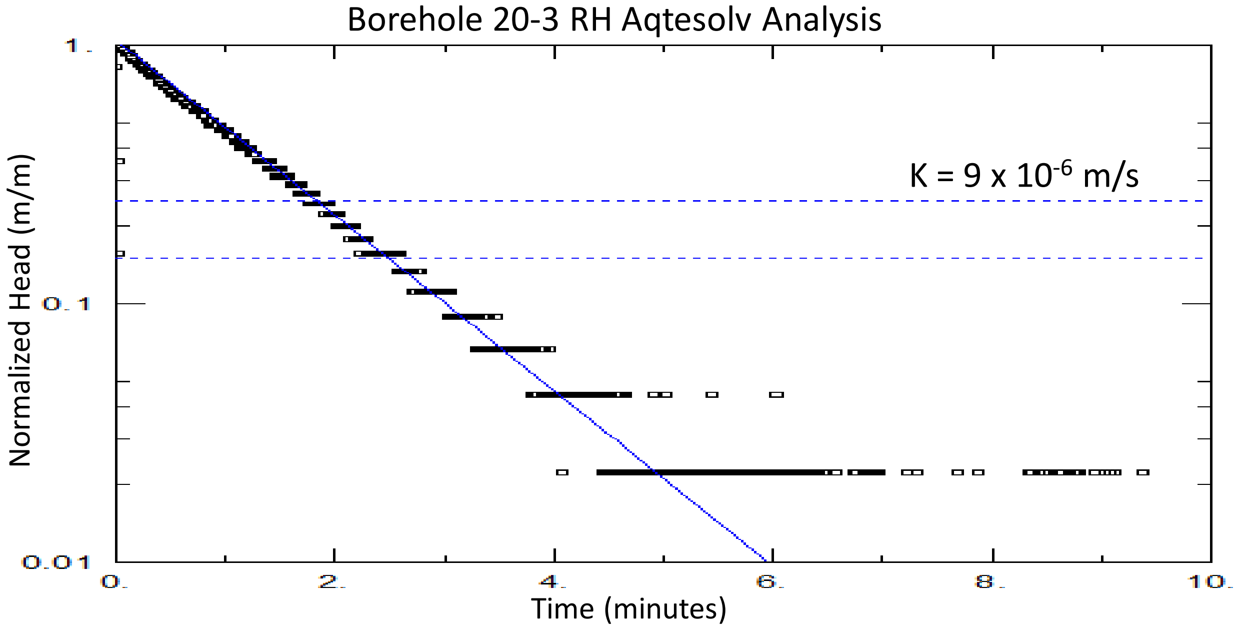
Date: July 2020

Project: 64742.02

Hydraulic Testing

FIGURE E4





Well Data:

Displacement observed (slug size): 0.37 metres (0.45 m)

Well Depth: 8.28 metres

Screen Length: 1.52 metres

Well Radius: 0.085 metres

Aquifer Data

Saturated Thickness: 1.52 metres

Anisotropy Ratio (Kz/Kr): 1

Aquifer Model: Confined, Hvorslev

Static Water Level: 1.12 metres bgs

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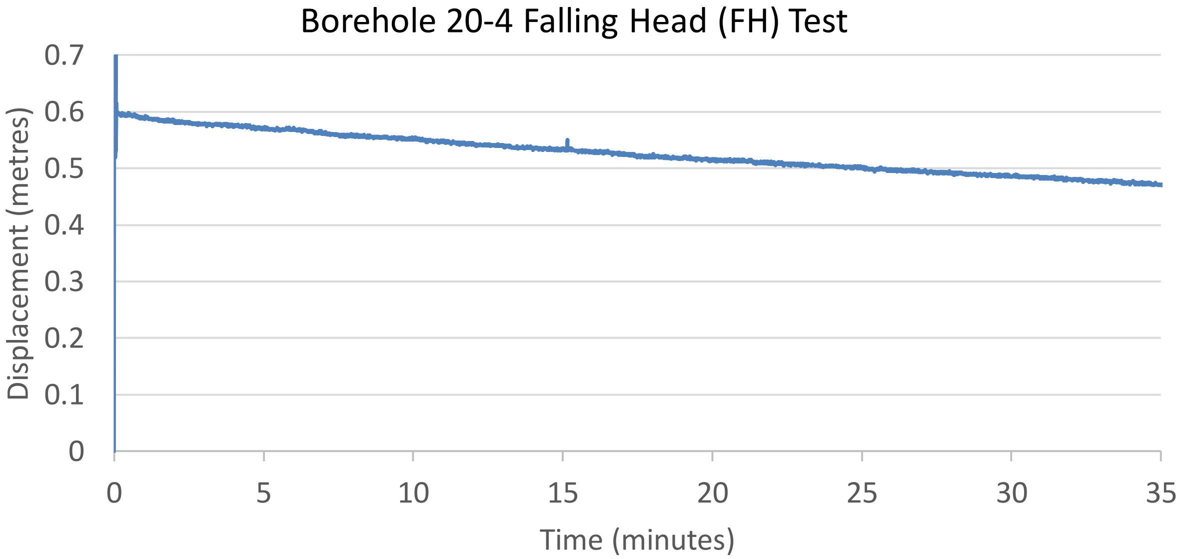
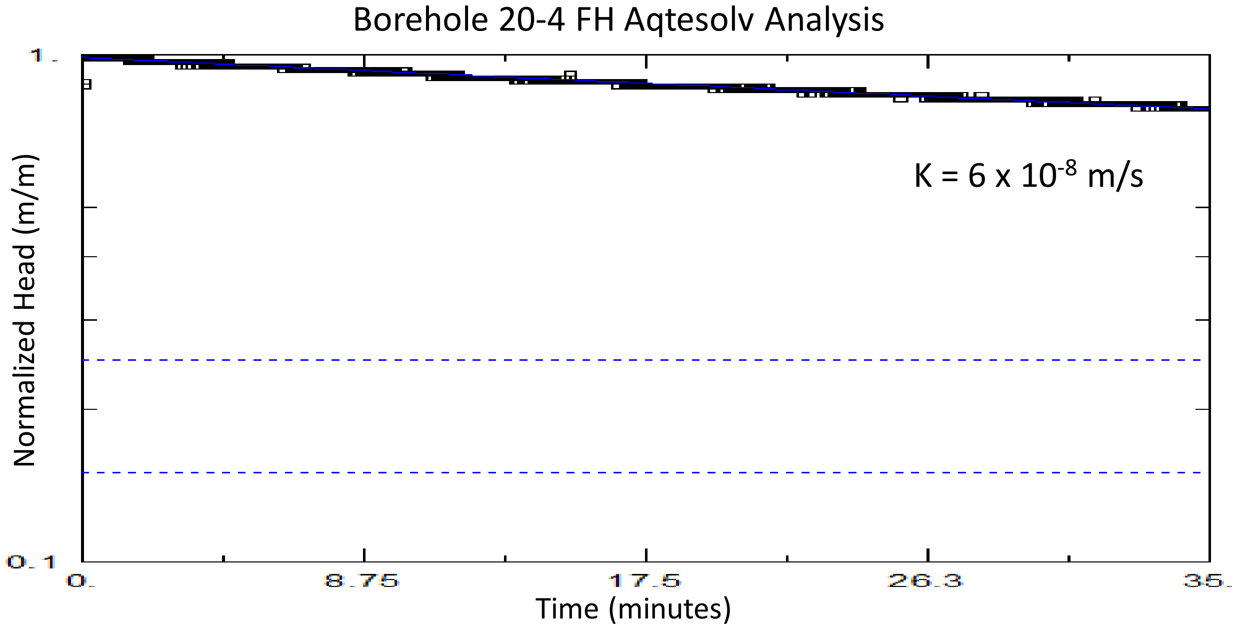
Hydraulic Testing

FIGURE E5





K = 6 x 10^{-8} m/s

Well Data:

Displacement observed (slug size): 1.73 metres (0.60 m)
Well Depth: 8.23 metres
Screen Length: 3.05 metres
Well Radius: 0.085 metres

Aquifer Data

Saturated Thickness: 3.05 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined, Hvorslev
Static Water Level: 1.04 metres bgs

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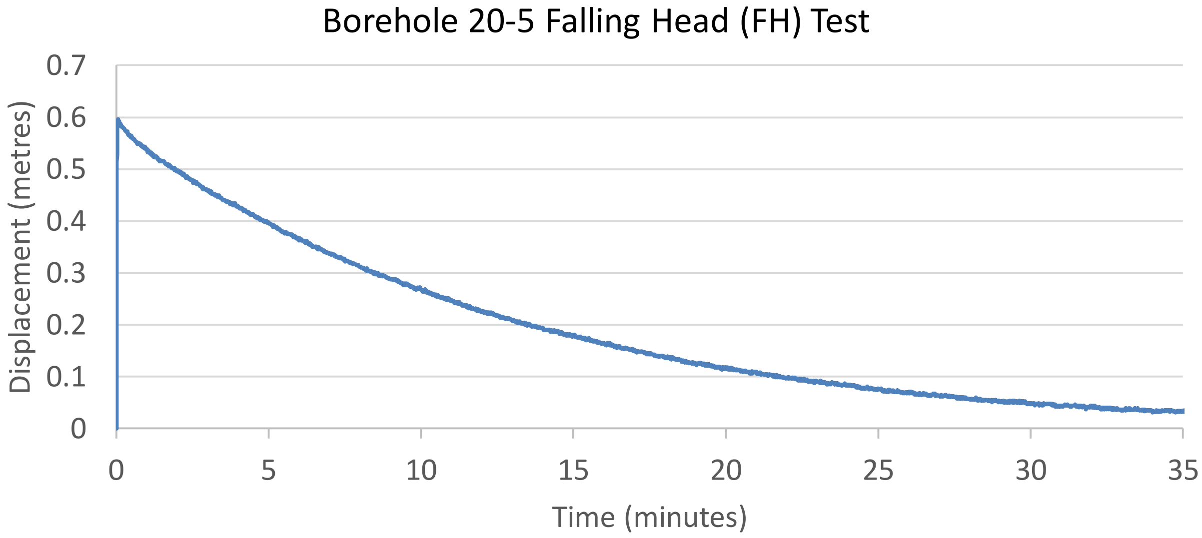
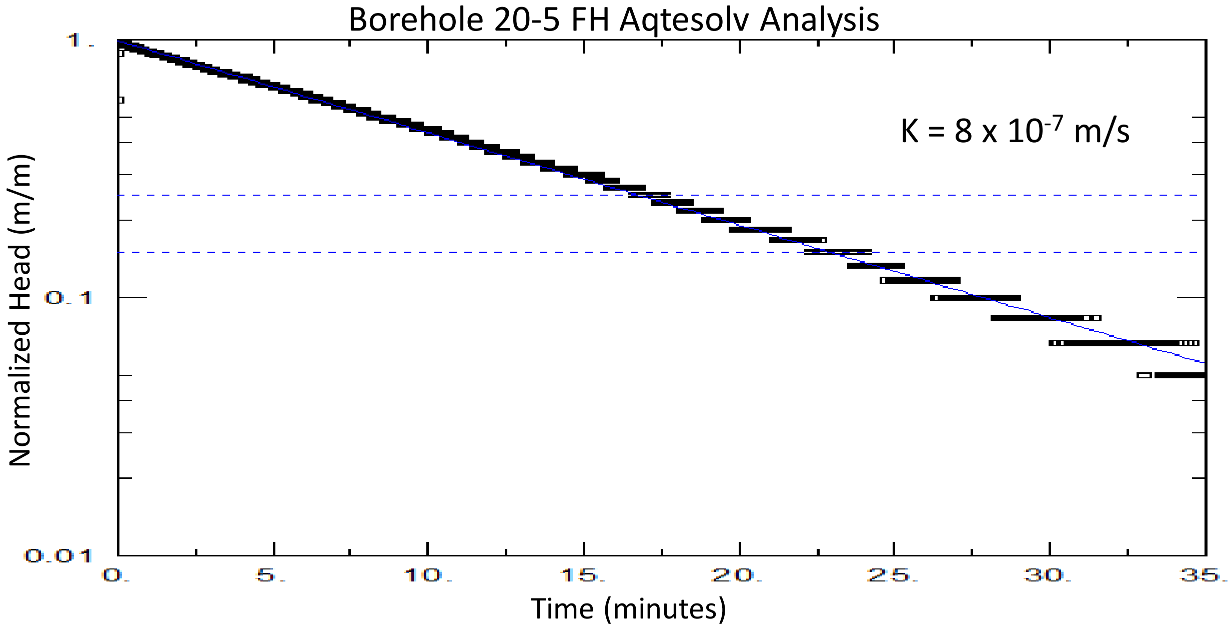
Hydraulic Testing

FIGURE E6





<u>Well Data:</u>

Displacement observed (slug size): 0.60 metres (0.60 m)
Well Depth: 9.35 metres
Screen Length: 3.05 metres
Well Radius: 0.085 metres

<u>Aquifer Data</u>

Saturated Thickness: 3.05 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined, Hvorslev
Static Water Level: 1.31 metres bgs

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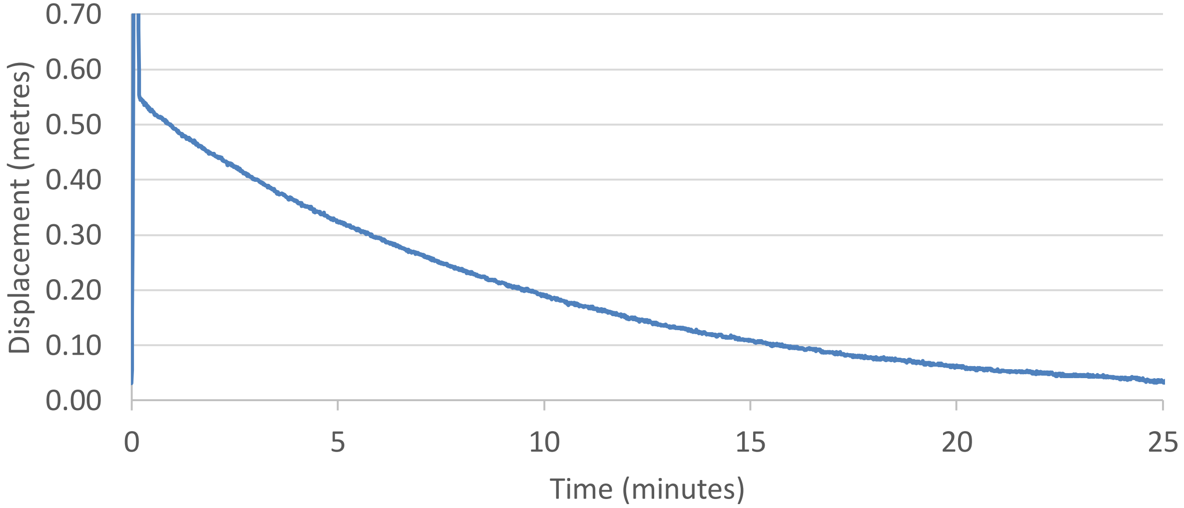
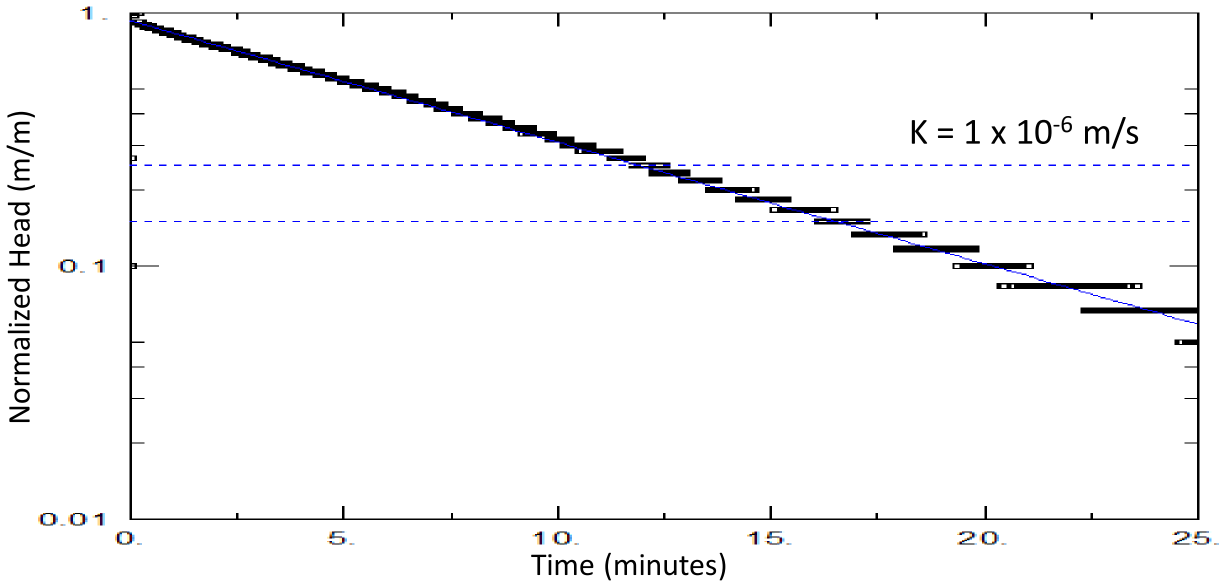
Hydraulic Testing

FIGURE E7

Borehole 20-5 Rising Head (RH) Test



Borehole 20-5 RH Aqtesolv Analysis



Well Data:
Displacement observed (slug size): 1.70 metres (0.60 m)
Well Depth: 9.35 metres
Screen Length: 3.05 metres
Well Radius: 0.085 metres

Aquifer Data
Saturated Thickness: 3.05 metres
Anisotropy Ratio (Kz/Kr): 1
Aquifer Model: Confined, Hvorslev
Static Water Level: 1.31 metres bgs

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Date: July 2020

Project: 64742.02

APPENDIX F

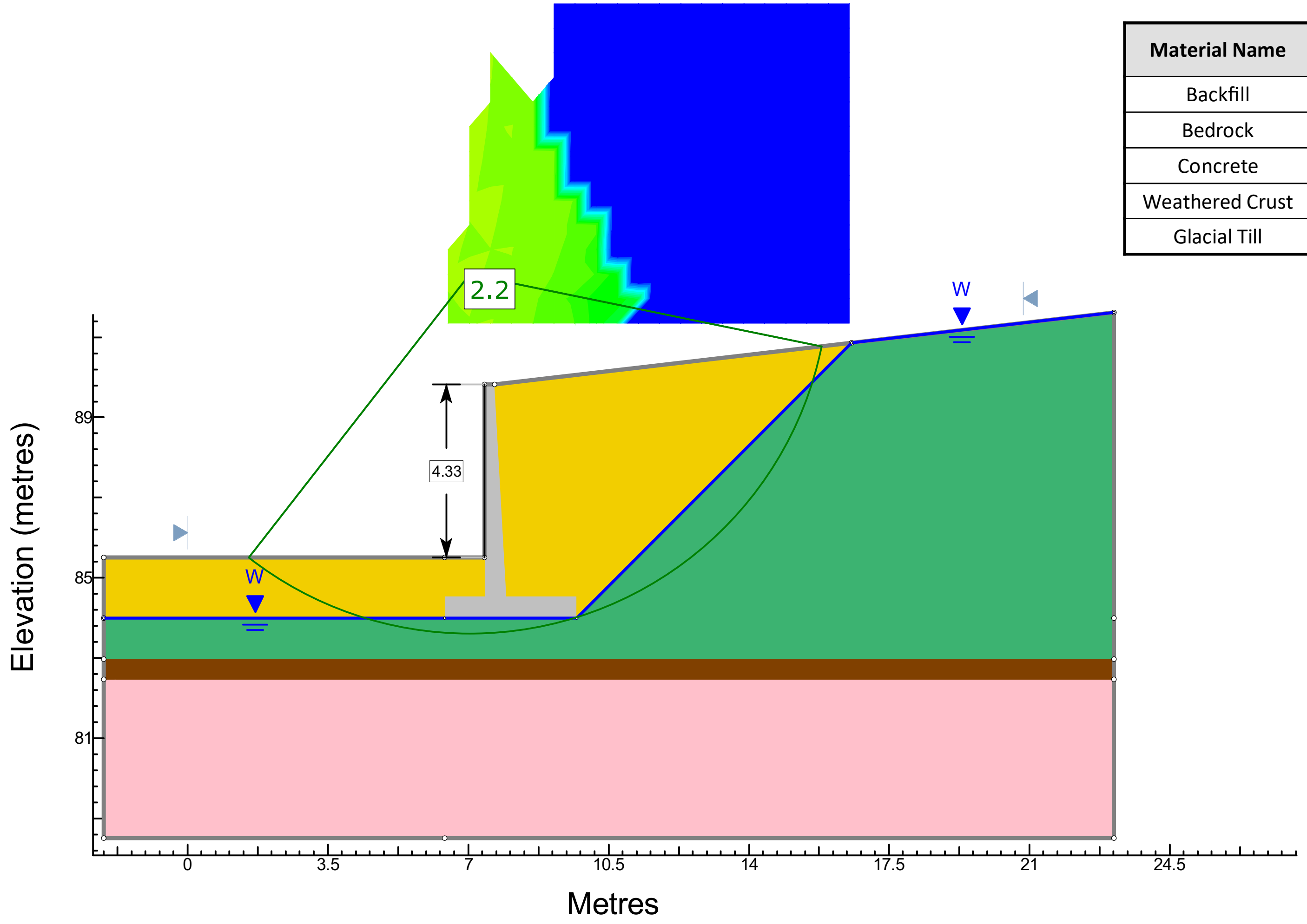
Global Stability Analysis of Retaining Wall
Figure F1 and F2

Report to: Megha Holdings Inc.
Project: 64742.02 (October 30, 2020) – R06

Figure F1

Static Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Backfill	🟨	21	Mohr-Coulomb	0	36
Bedrock	🟥	24	Infinite strength		
Concrete	⬜	24	Infinite strength		
Weathered Crust	🟩	18	Mohr-Coulomb	10	30
Glacial Till	🟫	20	Mohr-Coulomb	0	38



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SLIDEINTERPRET 8.022

Project
1243 Teron Road - Proposed Retaining Walls

Analysis Description
Static Conditions

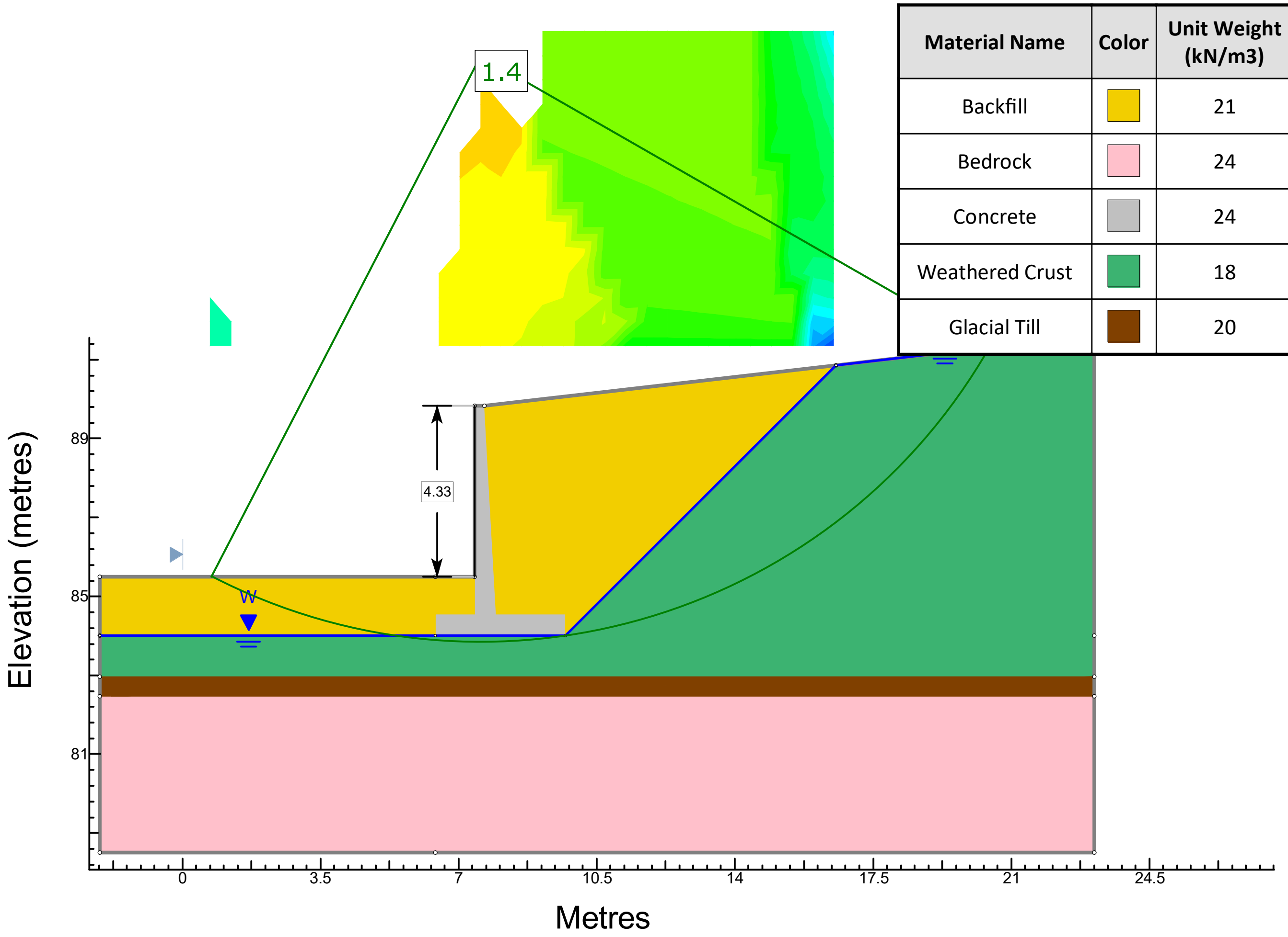
Drawn By G.D.
Scale 1:175
Company GEMTEC

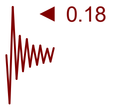
Date 07/16/2020
64742.02_Retaining Wall_Highest_2020-07-17.slim

Figure F2

Seismic Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Backfill	🟨	21	Mohr-Coulomb	0	36
Bedrock	🟥	24	Infinite strength		
Concrete	⬜	24	Infinite strength		
Weathered Crust	🟩	18	Mohr-Coulomb	10	30
Glacial Till	🟫	20	Mohr-Coulomb	0	38





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SLIDEINTERPRET 8.022

Project	1243 Teron Road - Proposed Retaining Walls
Analysis Description	Seismic Conditions
Drawn By	G.D.
Scale	1:184
Company	GEMTEC
Date	09/25/2020
	64742.02_Retaining Wall_Highest_V02_2020-07-17.slim

experience • knowledge • integrity

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

expérience • connaissance • intégrité