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Geotechnical Investigation
Proposed Commercial Development
5506 Manotick Main Street

Ottawa, Ontario

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Submitted to:

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7116 Bank Street
Metcalfe, Ontario
K0A 2P0
c/o Cedar Sands Holdings Inch.
184 Redpath Drive
Ottawa, Ontario
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Geotechnical Investigation
Proposed Commercial Development
5506 Manotick Main Street
Ottawa, Ontario

September 1, 2020
Project: 65032.03

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Attention: Mr. Steven Horvath

Re: Geotechnical Investigation
Proposed Commercial Development
5506 Manotick Main Street
Ottawa, Ontario

Enclosed is our geotechnical investigation for the above noted project, in accordance with our proposal dated November 29, 2019. This report was prepared by Mr. Alex Meacoe, P.Eng., and reviewed by Mr. John Cholewa, Ph.D, P.Eng.

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Enclosures
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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of a proposed commercial development located at 5506 Manotick Main Street in Ottawa, Ontario. The purpose of the investigation was to identify the general subsurface and groundwater conditions at the site by means of a limited number of boreholes. Based on the factual information obtained, preliminary engineering guidelines were to be provided on the geotechnical aspects of the design of the proposed development, including construction considerations that could influence design decisions.

This investigation was carried out in general accordance with our proposal dated November 29, 2019.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Plans are being prepared to construct a two-storey commercial building at 5506 Manotick Main Street in Ottawa, Ontario. The following is known about the site and project:

- The site is located at the south east corner of Manotick Main Street and Highcroft Drive;
- The site is currently occupied with an abandoned one-storey commercial building with at grade parking at the rear of the building;
- The proposed commercial building, which is to be located adjacent to Manotick Main Street, will be two-stories in height with one basement level, and dimensions of about 26 metres by 10 metres, in plan; and,
- At grade parking will be located at the rear of the building with dimensions of about 29 metres by 25 metres, in plan.

2.2 Review of Geology Maps

Based on our previous experience in the area of the site and surficial geology maps of the Ottawa area (Urban Geology Database of Canada's National Capital Region, Geological Survey of Canada, Open File 2878, 1994) the subsurface conditions at the site likely consist of silty clay over glacial till. Bedrock geology maps of the area show that the overburden deposits are underlain by dolostone of the Oxford formation. Fill material associated with the existing development of the site should be anticipated.

3.0 SUBSURFACE INVESTIGATION

The fieldwork for this investigation was carried out between October 16 and 18, 2019. During that time, three boreholes (numbered 19-1, 19-2, and 19-3) were advanced at the approximate locations shown on the Borehole Location Plan, Figure 1.

Boreholes 19-1 and 19-2 were advanced using a truck mounted, hollow stem auger drill rig supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of about 16.2 and 5.9 metres below ground surface, respectively. Practical auger refusal was encountered in borehole 19-1 and wash boring techniques were used to advance through the overburden. The abbreviation “DD” on the borehole logs refers to “diamond drilling” where wash boring and rotary diamond drilling techniques were used to advance past the cobbles and boulders present in the glacial till deposit.

Upon reaching the bedrock surface in borehole 19-1, the borehole was advanced into the bedrock using rotary diamond drilling techniques while retrieving HQ sized bedrock core.

Borehole 19-3 was advanced using portable drilling equipment supplied and operated by GEMTEC personnel. The portable drilling was advanced by advancing a drive open sampler within an open and uncased borehole using the one third weight hammer (about 23 kilograms). The borehole was advanced to a depth of about 1.7 metres below ground surface.

Standard penetration tests were carried out in the boreholes and samples of the soils encountered were recovered using a 50 millimetre diameter drive open sampler. The standard penetration tests were carried out in general conformance with ASTM S2488.

There is no correlation between shear strength and blow counts for unweathered Champlain Sea Clay for blow counts less than 2 blows. Silty clay with a blow count greater than about 2 or 3 blows in the weathered crust will generally have a shear strength of greater than 100 kilopascals, which is not measurable using standard Ministry of Transportation of Ontario (MTO) N-vane, and therefore, the shear strength of the weathered silty clay crust is conservatively assumed based on the measured standard penetration testing completed.

Well screens were installed in boreholes 19-1 and 19-2, to measure the groundwater levels. The groundwater levels were measured on January 7, 2020.

One soil sample recovered from borehole 19-2 was sent to Paracel Laboratories Ltd. for basic chemical testing relating to corrosion of buried concrete and steel.

Following the borehole drilling fieldwork, the soil samples were returned to our laboratory for examination by the geotechnical engineer and for geotechnical laboratory testing. Selected samples of the soil were tested for Atterberg Limit, water content, and grain size distribution testing.

The results of the boreholes are provided on the Record of Borehole sheets in Appendix A. The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 1. The results of the laboratory classification tests on the soil samples are provided in Appendix B. Photographs of the bedrock core samples are provided in Appendix C. The results of the chemical analysis of a sample of soil relating to corrosion of buried concrete and steel 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 borehole locations were determined using a Trimble R10 GPS. The elevations are referenced to geodetic datum NAD83 (CSRS) Epoch 2010, vertical network CGVD1928.

4.0 SUBSURFACE CONDITIONS

4.1 General

As previously indicated, the soil and groundwater conditions identified in the boreholes are given on the Record of Borehole Sheets (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, 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 augerholes. 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 groundwater conditions described in this report refer only to those observed at the place and time of observation noted in the report. These conditions may vary seasonally or as a consequence of construction activities in the area.

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

4.2 Existing Pavement Structure

Boreholes 19-1 and 19-2 were advanced through the existing at grade parking lot and driving lane, respectively, at the site. The pavement structure consists of about 10 and 40 millimetres of asphaltic concrete over about 390 and 240 millimetres of sand and gravel base layer in boreholes 19-1 and 19-2, respectively.

4.3 Silty Clay

Native deposits of silty clay were encountered below the pavement structure at boreholes 19-1 and 19-2, and at ground surface at borehole 19-3. The full thickness of the silty clay encountered in the boreholes has been weathered to a grey brown crust. The silty clay extends to depths of about 3.8 and 5.3 metres below ground surface in boreholes 19-1 and 19-2, respectively (elevations of about 84.0 and 83.0 metres). The silty clay was not fully penetrated in borehole 19-3, but was proven to a depth of about 1.7 metres below ground surface (elevation of about 86.0 metres).

Standard penetration tests carried out in the weathered crust gave N values ranging from 4 to 22 blows per 0.3 metres of penetration, which reflect a stiff to very stiff consistency. The upper portion of the silty clay at this site (and in the Ottawa area) has been weathered to a grey brown crust. There is no correlation between shear strength and blow counts for unweathered Champlain Sea Clay for blow counts less than 2 blows. Silty clay with a blow count greater than about 2 or 3 blows in the weathered crust will generally have a shear strength of greater than 100 kilopascals, which is not measurable using standard MTO N-vane. Therefore, it is conservative to assume a "stiff to very stiff" consistency in the weathered silty clay crust.

The results of Atterberg limit testing carried out on one sample of the weathered silty clay crust are provided on Plasticity Chart in Appendix B and are summarized in Table 4.1. The Atterberg limit testing was carried out in general conformance with ASTM D4318.

Table 4.1 – Summary of Atterberg Limit Testing (Weathered Silty Clay)

Borehole	Sample Number	Sample Depth (metres)	Water Content (%)	LL (%)	PL (%)	PI (%)
19-2	4	3.1 – 3.7	43	42	18	24

The measured water content of four samples of the weathered silty clay crust ranges from about 29 to 42 percent. The water content testing was carried out in general conformance with ASTM D4959.

4.4 Clayey Silt

A deposit of clayey silt with some sand and gravel was encountered below the silty clay in borehole 19-2 at a depth of about 5.3 metres below ground surface (elevation of about 83.0 metres). The clayey silt was not fully penetrated but was proven to about 5.9 metres below ground surface (elevation of about 82.4 metres).

The measured water content of one sample of the clayey silt was about 21 percent. The water content testing was carried out in general conformance with ASTM D4959.

4.5 Glacial Till

A native deposit of glacial till was encountered below the silty clay at borehole 19-1 at a depth of about 3.8 metres below ground surface (elevation of about 84.0 metres), and extends to a depth of about 11.6 metres below surface grade (elevation of about 76.3 metres). The glacial till is considered to be a heterogeneous mixture of all grain sizes, which at this site, can be described as grey brown to grey gravelly silty sand with trace clay. Practical auger refusal was encountered on cobbles and boulders within the glacial till deposit and wash boring techniques were required to advance through the glacial till deposit.

Standard penetration tests carried out within the glacial till gave N values ranging from 10 to greater than 50 blows per 0.3 metres of penetration, which reflects a loose to very dense relative density. It is noted that the N values obtained in the glacial till from standard penetration testing may have been impacted by cobble and boulder obstructions.

One grain size distribution test was undertaken on a sample of the glacial till from borehole 19-1. The results are provided in Appendix B and are summarized in Table 4.2. The grain size distribution testing was carried out in general conformance with ASTM D6913 and ASTM D7928.

Table 4.2 – Summary of Grain Size Distribution Test (Glacial Till)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
19-1	6	3.8 – 4.4	22	48	20	10

The moisture content of one sample of the glacial till was about 14 percent. The water content testing was carried out in general conformance with ASTM D4959.

4.6 Bedrock

Grey limestone bedrock with trace calcite deposits was encountered in borehole 19-1 at a depth of about 11.6 metres below ground surface (elevation of about 76.3 metres) and cored using rotary diamond drilling techniques while retrieving HQ sized bedrock core. The bedrock was cored to a depth of about 16.2 metres below ground surface (elevation of about 71.7 metres).

The recovered bedrock core samples have solid core recovery (SCR) values ranging from about 81 to 100 percent, and rock quality designation (RQD) values ranging from about 69 to 100 percent. Based on these values, the bedrock quality is considered to be fair to excellent.

Photographs of the bedrock core are presented on Figure C1 in Appendix C.

4.7 Groundwater Levels

Monitoring wells were installed in boreholes 19-1 and 19-2 to measure stabilized groundwater conditions. Table 4.3 summarizes the groundwater levels observed on January 7, 2020.

Table 4.3 – Summary of Groundwater Levels

Borehole	Well Screen	Ground Surface Elevation (metres)	Groundwater Depth (metres)	Groundwater Elevation (metres)
19-1	Bedrock	87.9	3.8	84.4
19-2	Silty Clay	88.3	2.2	86.1

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.

4.8 Soil Chemistry Relating to Corrosion

The results of chemical testing on a soil sample recovered from borehole 19-2 are provided in Appendix D and are summarized in Table 4.4 below.

Table 4.4: Summary of Corrosion Testing

Parameter	Borehole 19-2 Sample No. 2
Chloride Content (µg/g)	46
Resistivity (Ohm.m)	66.5
pH	7.4
Sulphate Content (µg/g)	13

5.0 RECOMMENDATIONS

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of 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 off site sources are outside the terms of reference for this report.

GEMTEC has conducted a Phase One and Phase Two Environmental Site Assessment for this property, which are provided in separate reports.

5.2 Excavation

The excavations for the proposed commercial development will be carried out through the topsoil, fill material and into the weathered silty clay crust deposit. The sides of the excavations should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. According to the Act, the overburden soils at this site can be classified as Type 3 and, accordingly, allowance should be made for excavation side slopes of 1 horizontal to 1 vertical, or flatter, excavation slopes for soils above the groundwater level.

Based on the measured groundwater elevations, excavation below the groundwater level as part of the development is not anticipated. Excavation of the native overburden deposits above the groundwater level should not present significant constraints.

The weathered silty clay crust deposit is sensitive to disturbance from ponded water, vibration and construction traffic. As such, it is suggested that final trimming to subgrade level be carried out using a hydraulic shovel equipped with a flat blade bucket. Allowance should be made to remove and replace any disturbed silty clay with compacted sand and gravel, such as that meeting OPSS Granular A or Granular B Type II, where required.

5.3 Groundwater Management

The groundwater levels on January 7, 2020 were measured to be about 3.5 and 2.2 metres below ground surface in boreholes 19-1 and 19-2, respectively.

Any groundwater inflow into the excavation should be handled from within the excavation by pumping from filtered sumps. Suitable detention and filtration will be required before discharging the water to a sewer or ditch. The amount of water entering the excavation for the construction of the foundations at this site should not exceed 50,000 litres per day and therefore it is not anticipated that an Environmental Activity and Sector Registry (EASR) will be required.

5.4 Foundation Design

Based on the results of the investigation, the proposed commercial development could be founded on footings bearing on or within the native undisturbed weathered silty clay crust deposits. The topsoil and fill material are considered to be highly compressible and should be removed from below any foundations and slabs on grade.

Based on plans provided, the proposed commercial building will be partially or fully located within the footprint of the existing house on site. Although not directly encountered, or sampled, during the drilling fieldwork, a layer of fill material of unknown composition associated with the construction of the existing house on site will be located surrounding the house to a depth of up to about 2.5 metres below ground surface. As such, the existing foundation elements and fill material associated with the past construction of the house will need to be removed from the proposed building area.

After the removal of the existing house and associated fill material, and where the existing subgrade surface is below the proposed founding level, the grade could be raised with compacted granular material (engineered fill) with a Class II non-woven geotextile having an FOS not exceeding 100 microns (OPSS 1860) placed on the subgrade. 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 0.5 metres beyond the footings and then down and out from this point at 1 horizontal to 1 vertical, or flatter.

For design purposes, exterior footings bearing on the native, undisturbed weathered silty clay crust, or on a pad of engineered fill above native, undisturbed weathered silty clay crust should be sized using a geotechnical reaction at Serviceability Limit State (SLS) of 100 kilopascals and a factored geotechnical resistance at Ultimate Limit State (ULS) of 300 kilopascals.

The post construction total and differential settlement of the footings at SLS should be less than 25 and 15 millimetres, respectively, provided that all loose or disturbed soil is removed from the bearing surfaces.

To reduce the potential for cracking in the footings, foundation walls, and concrete slab on grade where the footings transition between different subgrade materials, the foundation walls should be reinforced for a distance of 3 metres on both sides of the transition areas or as recommended by the structural engineer.

5.5 Grade Raise Restrictions

The site is underlain by native deposits of stiff to very stiff weathered silty clay crust over glacial till. Based on the borehole information, there are no grade raise restrictions at this site, from a geotechnical perspective. The settlement due to compression of the native soils due to fill placement should be relatively small and should occur during or shortly after the fill placement.

5.6 Frost Protection of Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) footings that are located in areas that are to be cleared of snow should be provided with at least 1.8 metres of earth cover for frost protection purposes. 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.

If the foundation and/or slab on grade are insulated in a manner that will reduce heat flow to the surrounding soil, the foundation depth shall conform to that required for foundations for an unheated space.

5.7 Seismic Design of Proposed Structures

Based on the results of the investigation, it is anticipated that the proposed foundations will be supported on a deposit of stiff to very stiff weathered silty clay crust or a pad of engineered fill constructed on the weathered crust.

Based on Table 4.1.8.4.A., the seismic site class can be determined based on the Average Standard Penetration Resistance or the Soil Undrained Shear Strength. In the National Building Code of Canada, Commentary J, sentence 98, the soil strata can be separated into the two profiles, one for the silty clay and another for the silty sand (glacial till) and bedrock. It was conservatively assumed that the silty clay has an undrained shear strength of stiff to very stiff (50 kPa < s_u ≤ 100 kPa) corresponding to a Site Class D. The glacial till and bedrock has an average standard penetration resistance of about 52 blows (assuming an N value of 50 blows for the cobbles and boulders within the glacial till and the bedrock) which corresponds to a Site Class C.

The lowest site class of the two soil profiles will govern, and therefore, the overall site can be given a Site Class D.

There is no potential for liquefaction of the overburden deposits at this site.

5.8 Foundation Wall Backfill and Drainage

The native deposits at this site are frost susceptible and should not be used as backfill against foundations. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting the requirements of OPSS Granular A, or Granular B Type I or II.

Where the backfill will ultimately support areas of hard surfacing (pavement, sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Light walk behind compaction equipment should be used next to the 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 (concrete, sidewalks, 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 fill 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.

The frost susceptible native soils could be considered for foundation wall backfill purposes in landscaped areas provided that a suitable bond break is applied to the surface of the foundations to prevent frost jacking. A suitable bond break could consist of at least 2 layers of 6 MIL polyethylene sheeting or a proprietary plastic drainage medium. It is also pointed out that the native soils at this site can be impacted by changes in moisture content and this could affect the ability to compact this material to the required density.

Drainage of the foundation wall backfill should be provided by means of a perforated pipe subdrain in a surround of 19 millimetre clear stone, fully wrapped in geotextile, which leads by gravity drainage to an adjacent storm sewer.

5.9 Lateral Earth Pressures

Foundation walls that are backfilled with granular material such as that meeting OPSS Granular B Type I or II requirements should be designed to resist “at rest” earth pressures calculated using the following formula:

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

where;

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

Seismic shaking can increase the forces on the retaining wall. The total “At Rest” thrust acting on the walls (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: Total "At Rest" thrust (kN/m);
- 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 (m).

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_o) acts at a point located about 0.6H above the base of the wall.

For design purposes, the parameters provided in Table 5.1 can be used to calculate the thrust acting on the walls during static and seismic loading conditions.

Table 5.1 – Summary of Design Parameters (Building Foundation Walls)

Parameter	OPSS Granular B Type I	OPSS Granular B Type II
Material Unit Weight, γ (kN/m^3)	22	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	0.50[1]	0.43[1]

Notes:

1) According to the 2015 National Building Code of Canada, the peak ground acceleration (PGA) for this site is 0.28 for Site Class C. For this particular site, the corrected PGA can be taken as 0.30 g (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, kh, of 0.15 and assuming that the vertical seismic coefficient, kv, is zero.

Heavy construction traffic should not be allowed to operate adjacent to foundation walls for the proposed building (within about 2 metres horizontal) during construction, without the approval of the designers.

5.10 Basement Floor Slabs

As discussed in Section 5.4 above, the proposed building will be partially or fully located within the footprint of the excavation from the existing house and, as such, fill material associated with the construction of the existing house should be anticipated below the proposed slab on grade.

To provide predictable settlement performance of the basement slab, all fill material associated with the construction of the existing house, loose soil, or debris should be removed from the slab area. The base of the floor slab should consist of at least 200 millimetres of 19 millimetre clear crushed stone. Any necessary grade raise fill should consist of either 19 millimetre clear crushed stone or OPSS Granular B Type II. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular B Type II material. Since the source of recycled material cannot be determined or controlled, it is suggested that any imported Granular B Type II materials be composed of 100 percent crushed rock only.

The clear crushed stone should be nominally compacted in maximum 300 millimetre thick lifts with at least 2 passes of a diesel plate compactor. The Granular B Type II should be compacted in maximum 150 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment.

Underfloor drainage should be provided below the basement floor slab. If well graded granular material (such as OPSS Granular B Type II) is used below the basement floor slab, we suggest that drainage be provided by means of plastic perforated pipes spaced at about 6 metres horizontally or as required to link any hydraulically isolated areas in the basement. If clear crushed stone is used below the basement floor slab, drains are not considered essential provided that the clear stone can outlet to the sump and drains are installed to link any hydraulically isolated areas in the basement. The drains should outlet by gravity to a storm sewer.

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.

Proper moisture protection with a vapour retarder should be used for any slab on grade 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 floor slab.

5.11 Proposed Services

5.11.1 Excavation

In the overburden, the excavation for flexible service pipes should be in accordance with Ontario Provincial Standard Drawing (OPSD) 802.010 for Type 3 soil. The excavation for rigid service pipes should be in accordance with OPSD 802.031 for Type 3 soil. 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. As an alternative or where space constraints dictate, the service installations could be carried out within a tightly fitting, braced steel trench box, which is specifically designed for this purpose.

Groundwater seepage into excavations is expected and should be controlled, as necessary, by pumping from within the excavations. It is not expected that short term pumping during excavation will have a significant effect on nearby structures and services.

5.11.2 Pipe Bedding

The bedding for service pipes should be in accordance with OPSD 802.010 and 802.031 for flexible and rigid pipes in Type 3 soils, respectively. The bedding for service pipes should consist of at least 150 millimetres of crushed stone meeting OPSS requirements for Granular A.

Cover material, from spring line to at least 300 millimetres above the tops of the pipes, should consist of granular material, such as that meeting OPSS Granular A.

In areas where the subsoil is disturbed or where unsuitable material (such as fill or organic material) exists below the pipe subgrade level, the disturbed/unsuitable material should be removed and replaced with a subbedding layer of compacted granular material, such as that meeting OPSS Granular B Type I or II. To provide adequate support for the pipes in the long term in areas where subexcavation of material is required below design subgrade level, the excavations should be sized to allow a 1 horizontal to 1 vertical or 2 horizontal to 1 vertical spread of granular material down and out from the bottom of the pipes.

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 granular bedding and subbedding materials should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value.

The use of clear crushed stone as a bedding, subbedding or cover material should not be permitted on this project.

5.11.3 Trench Backfill

In areas where the service trench will be located below or in close proximity to existing or future areas of hard surfacing (pavement, sidewalk, etc.), acceptable native materials should be used as backfill between the roadway subgrade level and the depth of seasonal frost penetration in order to reduce the potential for differential frost heaving between the area over the trench and the adjacent hard surfaced area. The depth of frost penetration in exposed areas can normally be taken as 1.8 metres below finished grade. 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 or II..

To minimize future settlement of the backfill and achieve an acceptable subgrade for the parking areas, sidewalks, 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. 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.

5.12 Access Roadway/Parking Lot Areas

5.12.1 Subgrade Preparation

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

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

In the area of the existing house, and any other areas where it will 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 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.

5.12.2 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 (Two 40 millimetre lifts of Superpave 12.5), over
- 150 millimetres of OPSS Granular A base, over
- 300 millimetres of OPSS Granular B, Type II subbase

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 over 60 millimetres of Superpave 19.0), over
- 150 millimetres of OPSS Granular A base, over
- 450 millimetres of OPSS Granular B, Type II subbase

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

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, install a woven geotextile separator between the roadway subgrade surface and the granular subbase material, or a combination of both, to prevent pumping and disturbance to the subbase material. The contractor should be made responsible for their construction access.

5.12.3 Asphalt Cement Type

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

5.12.4 Pavement Transitions

As part of the access roadway/parking lot construction, the new pavement will abut the existing pavement at Highcroft Drive. 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.

5.12.5 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 subgrade surfaces should be crowned and shaped to drain to the ditches and/or catch basins to promote drainage of the pavement granular materials.

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

5.12.6 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.

5.13 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the sample of soil recovered from borehole 19-2 was 13 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.

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

6.0 ADDITIONAL CONSIDERATIONS

6.1 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction and excavation) 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. However, the magnitude of the vibrations is expected to be much less than that required to cause damage to the nearby structures or services.

6.2 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.

6.3 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.

6.4 Well Abandonment

The monitoring wells installed in boreholes 19-1 and 19-2 as part of this investigation should be decommissioned by a licensed well technician. The well abandonment could be carried out in advance of, or during the construction.

6.5 Design Review and Construction Observation

The final details for the proposed construction 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.

In accordance with Section 4.2.2.2 of the Ontario Building Code (2012), 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 structures, access roadways, and parking areas 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.

7.0 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.

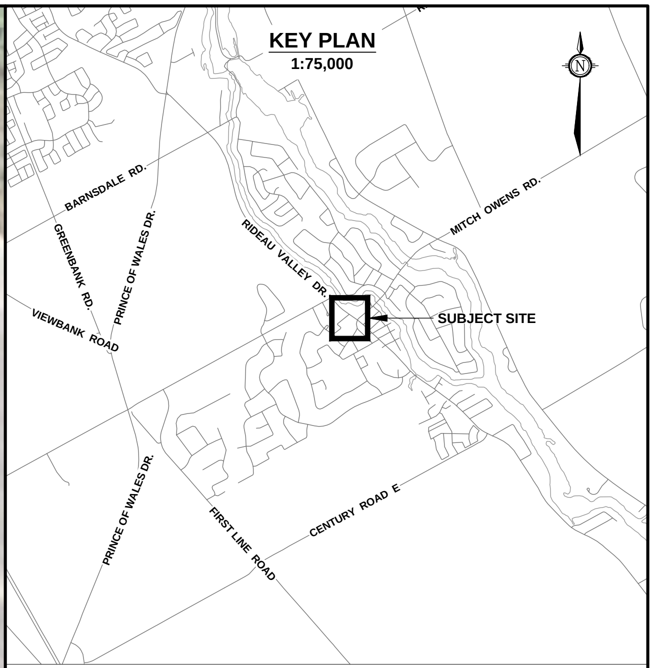
W. Alex Meacoe

Alex Meacoe, P.Eng.
Geotechnical Engineer

John Cholewa, Ph.D., P.Eng.
Senior Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
J. A. CHOLEWA
100119055
01 Sep 2020
PROVINCE OF ONTARIO

KEY PLAN
1:75,000



LEGEND

⊕ **BOREHOLE LOCATION**
(current investigation by GEMTEC)

⊕ **BOREHOLE WITH MONITORING WELL LOCATION**
(current investigation by GEMTEC)

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

NOTE:

ALL PROPOSED STRUCTURES HAVE BEEN ILLUSTRATED BY USING DRAWING SP1-0, SITE PLAN BY GRANT+HENLEY DESIGN GROUP. THUS, ALL PROPOSED STRUCTURES SHOWN ARE FOR ILLUSTRATION PURPOSES ONLY.

Scale 1:250

0 5 10 15m

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www.gemtec.ca
ottawa@gemtec.ca

Drawing
BOREHOLE LOCATION PLAN

Client
KGMS CONSTRUCTION

Project
65032.03

GEOTECHNICAL INVESTIGATION
5506 MANOTICK MAIN ST.
OTTAWA, ONTARIO

Drwn by
P.C.

Chkd by
W.A.M

Date
FEBRUARY 2020

Rev.
0

FIGURE 1

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APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Symbols

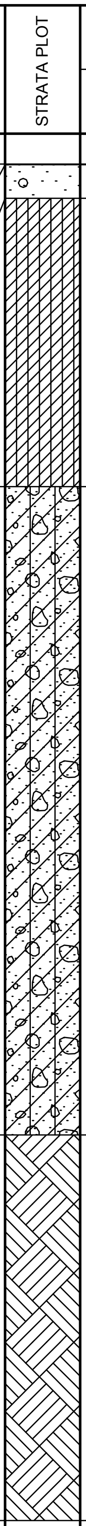
Report to: 2538702 Ontario Inc. o/a KGMS Construction
Project: 65032.03 (September 1, 2020)

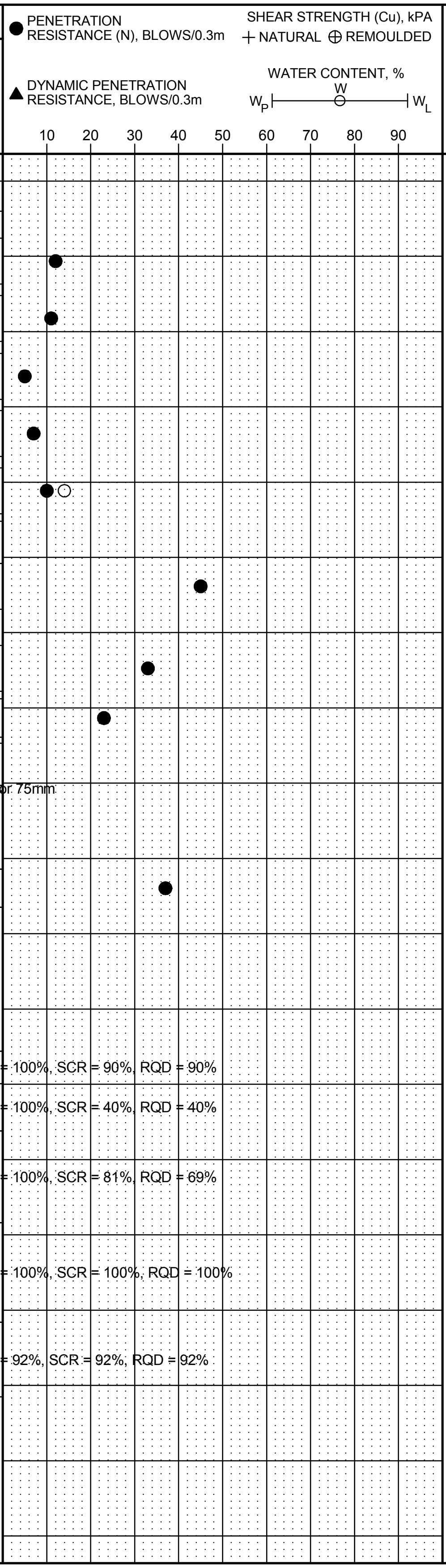
RECORD OF BOREHOLE 19-1

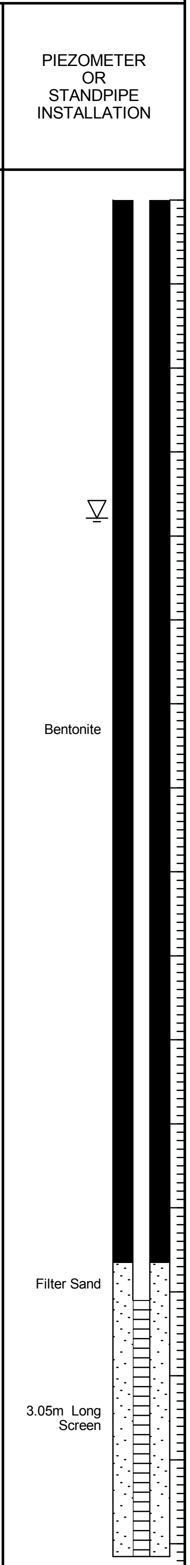
CLIENT: KGMS Construction
PROJECT: 5506 Manotick Main Street, Ottawa, Ontario
JOB#: 65032.03
LOCATION: See Figure 1, Borehole Location Plan

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

BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING
	Ground Surface	87.88					
Power Auger Hollow Stem Auger (200mm OD)	ASPHALTIC CONCRETE	0.01	1	AS			
	Brown sand and gravel, trace to some silt (BASE MATERIAL)	0.41					
	Stiff to very sitff, grey brown SILTY CLAY (WEATHER CRUST), contains silty sand with gravel		2	SS	430	12	
			3	SS	610	11	
			4	SS	610	5	
			5	SS	610	7	
	Loose to very dense, grey silty sand with some gravel, contains cobbles and boulders (GLACIAL TILL)	84.04 3.84	6	SS	585	10	MH
Wash Casing HQ Casing			7	RC	355	DD	
			8	SS	100	45	
			9	SS	50	33	
			10	SS	150	23	
			11	RC	555	DD	
			12	SS	75	>50 for 75mm	
			13	RC	485	DD	
			14	SS	250	37	
			15	RC	0	DD	
Diamond Rotary Core HQ (96mm OD)	Slightly weathered to fresh, thinly to medium bedded, LIMESTONE BEDROCK	76.32 11.56	16	RC		TCR = 100%, SCR = 90%, RQD = 90%	
			17	RC		TCR = 100%, SCR = 40%, RQD = 40%	
			18	RC		TCR = 100%, SCR = 81%, RQD = 69%	
			19	RC		TCR = 100%, SCR = 100%, RQD = 100%	
			20	RC		TCR = 92%, SCR = 92%, RQD = 92%	
	End of Borehole	71.73 16.15					







GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
20/01/07	3.8 ▽	84.1

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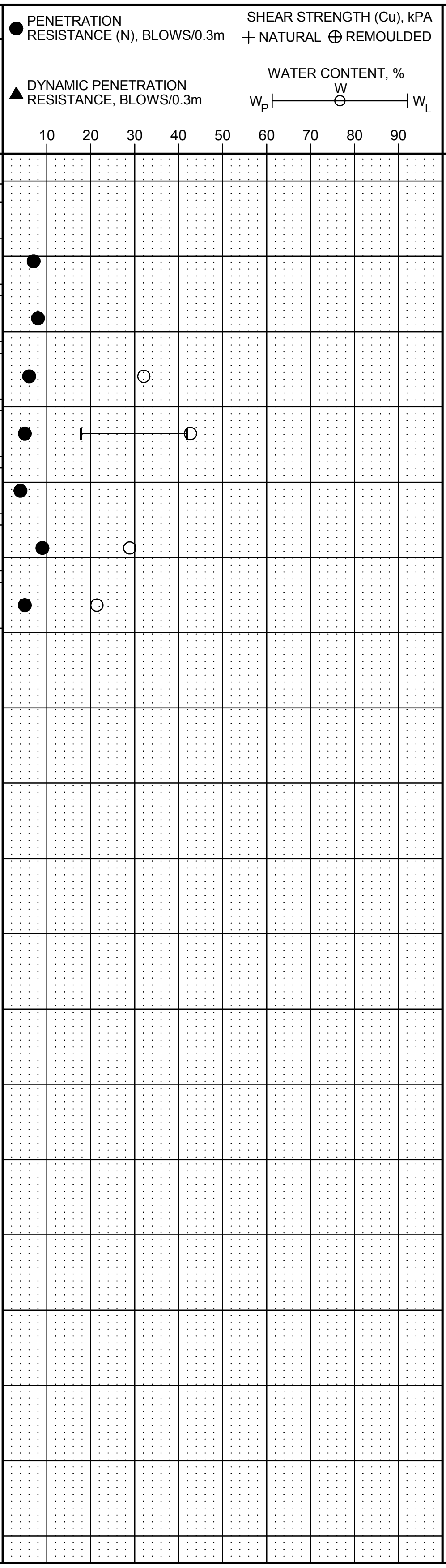
LOGGED: A.N.
CHECKED: A.W.M

RECORD OF BOREHOLE 19-2

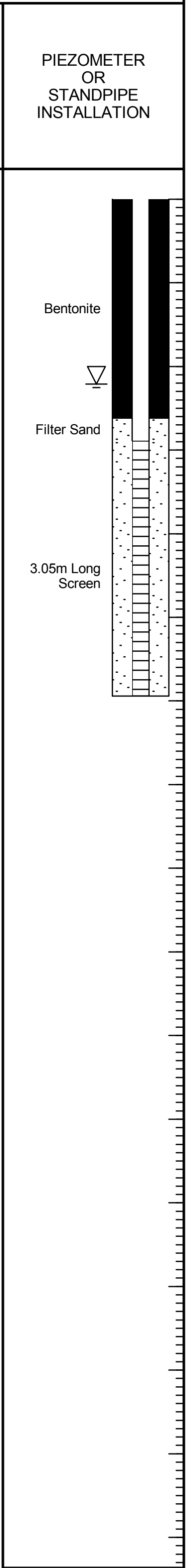
CLIENT: KGMS Construction
PROJECT: 5506 Manotick Main Street, Ottawa, Ontario
JOB#: 65032.03
LOCATION: See Figure 1, Borehole Location Plan

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Dec 17 2019

BORING METHOD	SOIL PROFILE			SAMPLES			
	DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m
Power Auger Hollow Stem Auger (200mm OD)	Ground Surface		88.32				
	ASPHALTIC CONCRETE		0.04	1a	GS		-
	Brown sand and gravel (BASE MATERIAL)		0.28				
	Stiff to very stiff, grey brown SILTY CLAY (WEATHERED CRUST)			1	SS	230	7
				2	SS	610	8
				3	SS	610	6
				4	SS	610	5
				5	SS	610	4
				6	SS	460	9
	Stiff, grey CLAYEY SILT, some sand and gravel		82.99 5.33	7	SS	230	5
	End of Borehole		82.38 5.94				



ADDITIONAL LAB. TESTING



GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
20/01/07	2.2 ▽	86.1

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LOGGED: N.S.

CHECKED: A.W.M

RECORD OF BOREHOLE 19-3

CLIENT: KGMS Construction
PROJECT: 5506 Manotick Main Street, Ottawa, Ontario
JOB#: 65032.03
LOCATION: See Figure 1, Borehole Location Plan

SHEET: 1 OF 1
DATUM: CGVD28
BORING DATE: Dec 18 2019

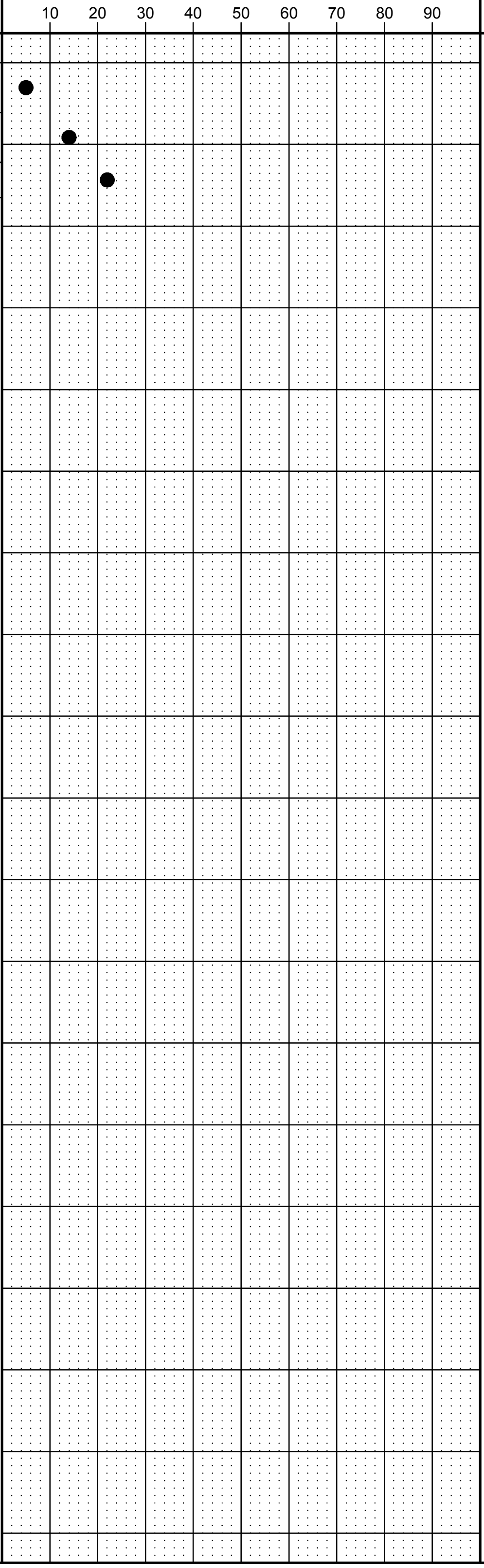
DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	RECOVERY, mm	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Portable Drill Rig Open Hole	Ground Surface		87.67						
		Brown silty clay with sand (FILL MATERIAL)	▨	86.88	1	SS	230	5		Native Backfill
1		Stiff to very stiff, grey brown SILTY CLAY (WEATHERED CRUST)	▨	0.79	2	SS	355	14		
				86.02	3	SS	430	22		
		End of Borehole		1.65						
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										
16										
17										
18										

● 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



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LOGGED: N.S.

CHECKED: A.W.M

ABBREVIATIONS AND TERMINOLOGY USED ON RECORDS OF BOREHOLES AND TEST PITS

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

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

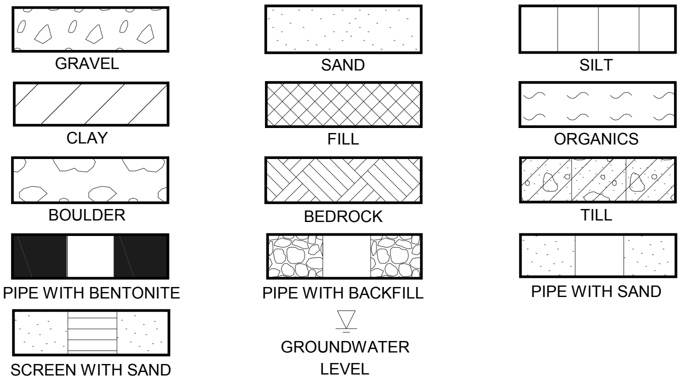
PENETRATION RESISTANCE

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

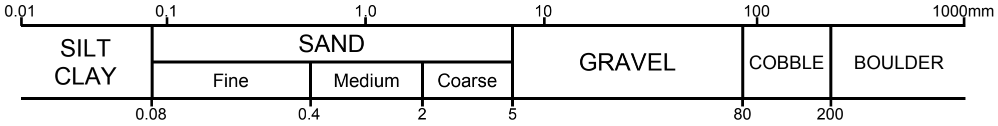
Dynamic Penetration Resistance
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive a 50 mm (2 in.) diameter 60° cone attached to 'A' size drill rods for a distance of 300 mm (12 in.).

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

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



GRAIN SIZE



DESCRIPTIVE TERMINOLOGY

(Based on the CANFEM 4th Edition)

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

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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

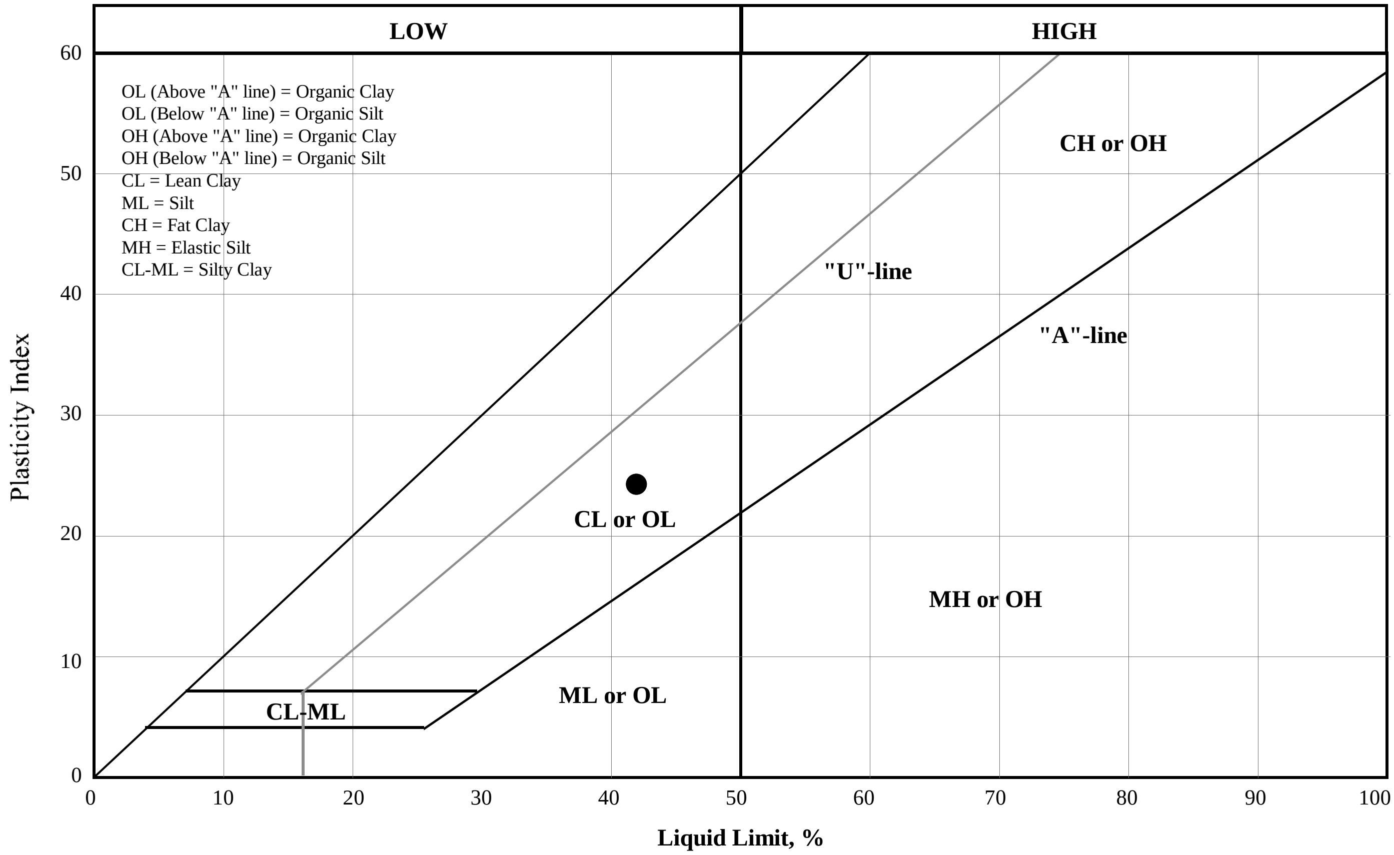
ROCK QUALITY	
RQD	**Overall Quality**
0 - 25	Very poor
25 - 50	Poor
50 - 75	Fair
75 - 90	Good
90 - 100	Excellent

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

APPENDIX B

Laboratory Test Results

GEMTEC Consulting Engineers and Scientists	Client: KGMS Construction	Plasticity Chart
	Project: Geotechnical and Environmental Investigations	
	Project #: 6503203	

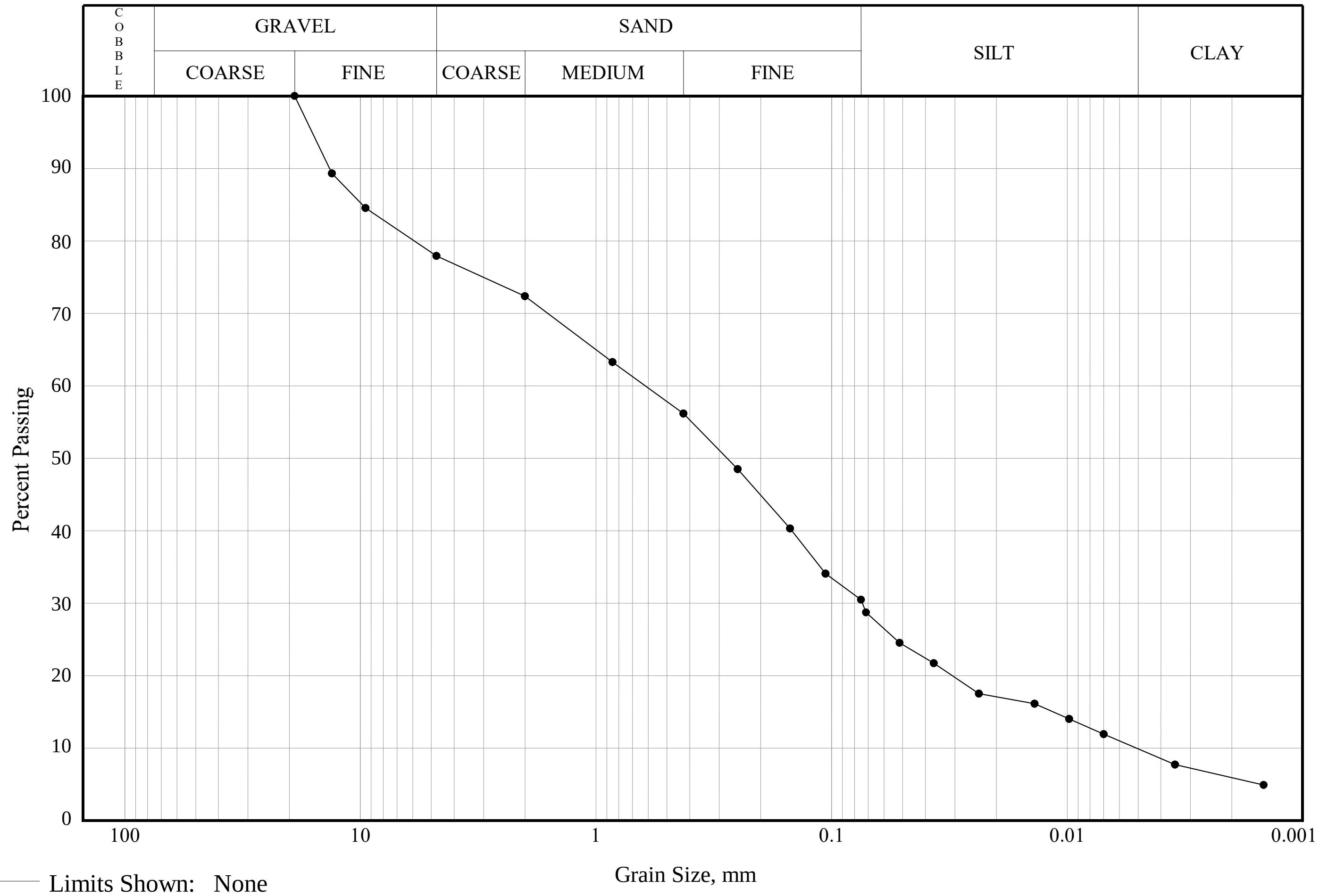


Symbol	Borehole /Test Pit	Sample Number	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Non-Plastic	Moisture Content, %
●	BH 19-02	04	3.05-3.66	41.9	17.7	24.2	☐	42.69

GEMTEC
Consulting Engineers and Scientists

Client:	KGMS Construction
Project:	Geotechnical and Environmental Investigations, Erosion
Project #:	6503203

Soils Grading Chart



Limits Shown: None

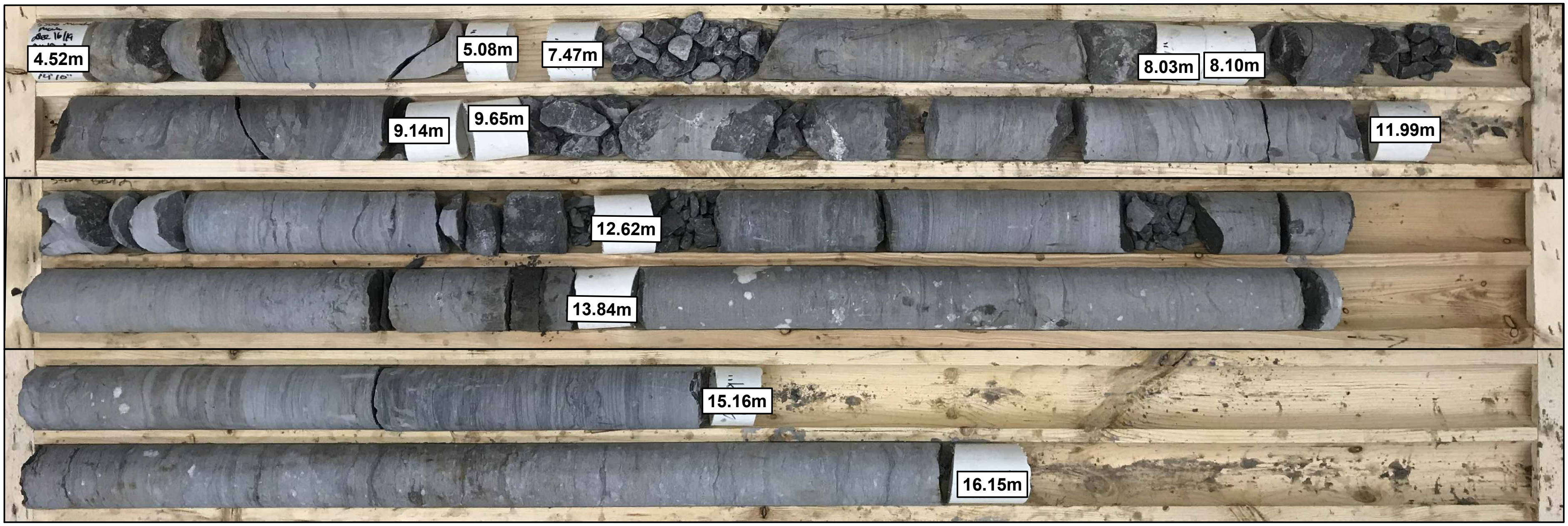
Line Symbol	Sample	Borehole/ Test Pit	Sample Number	Depth	% Cob.+ Gravel	% Sand	% Silt	% Clay
—●—	Glacial Till	BH 19-01	06	3.81-4.42	22.1	47.5	20.6	9.9

Line Symbol	CanFEM Classification	USCS Symbol	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}	% 5-75μm
—●—	Gravelly silty sand , trace clay	N/A	0.01	0.01	0.07	0.28	0.62	9.82	20.6

APPENDIX C

Rock Core Photographs
Bedrock Description Terminology

BOREHOLE 19-1
BORING DATE: DECEMBER 16, 2019
DEPTH: 4.52 to 16.15 mbgs



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Project

COMMERCIAL DEVELOPMENT
566 MANOTICK MAIN
OTTAWA, ONTARIO

FIGURE C1

File No.
65032.03

ROCK CORE PHOTOGRAPH
BOREHOLE 19-1

APPENDIX D

Chemical Analysis of Soil Sample
Samples Relating to Corrosion
(Paracel Laboratories Ltd. Order No. 2002044)

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 10-Jan-2020
Order Date: 6-Jan-2020
Project Description: 65032.03

	Client ID:	BH19-2 SA 2	-	-	-
	Sample Date:	19-Dec-19 09:00	-	-	-
	Sample ID:	2002044-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	63.3	-	-	-
General Inorganics					
Conductivity	5 uS/cm	150	-	-	-
pH	0.05 pH Units	7.40	-	-	-
Resistivity	0.10 Ohm.m	66.5	-	-	-
Anions					
Chloride	5 ug/g dry	46	-	-	-
Sulphate	5 ug/g dry	13	-	-	-

experience • knowledge • integrity

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

expérience • connaissance • intégrité