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Geotechnical Investigation
Proposed School Development
2405 & 2419 Mer Bleue Road
Ottawa, Ontario

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Submitted to:

Conseil des écoles publiques de l'Est de l'Ontario
Service des immobilisations
2445 St. Laurent Boulevard
Ottawa, Ontario
K1G 6C3

Geotechnical Investigation
Proposed School Development
2405 & 2419 Mer Bleue Road
Ottawa, Ontario

May 3, 2018
Project: 62721.07

experience • knowledge • integrity

GEMTEC Consulting Engineers and Scientists Limited
32 Steacie Drive
Ottawa, ON, Canada
K2K 2A9

May 3, 2018 File: 62721.07

Conseil des écoles publiques de l'Est de l'Ontario
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2445 St. Laurent Boulevard
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Attention: Mr. Benoit Duquette

Re: Geotechnical Investigation
Proposed School Development
2405 & 2419 Mer Bleue Road
Ottawa, Ontario

Please find enclosed the report summarizing the findings of our geotechnical investigation for the proposed school development at 2405 & 2419 Mer Bleue Road in Ottawa, Ontario.

Kelsey Holkestad, B.Eng, E.I.T. John Cholewa, Ph.D., P.Eng.

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation conducted by GEMTEC Consulting Engineers and Scientists limited (GEMTEC) carried out at the site of a proposed school development located at 2405 and 2419 Mer Bleue Road in Ottawa, Ontario. 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 accordance with our proposals dated February 9, 2018 and March 29, 2019.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Plans are being prepared to construct an elementary school and a secondary school on two combined lots located at 2405 and 2419 Mer Bleue Road in Ottawa, Ontario (see Key Plan, Figure 1). The two lots will be treated as a single site, which has a total area of approximately 12 acres. It is understood that the elementary school will be located in the southwest quadrant of the property and the high school will be located in the northeast quadrant of the property. The elementary school will be two to three storeys in height and will have a total area of 5,251 square metres. The high school will be three to four storeys and will have a total area of 5,971 square metres. Details on the foundations of the schools was not available at the time of this report. For the purpose of this report, it is assumed that the buildings will consist of slab on grade construction.

2.2 Review of Geology Maps

Based on available surficial geology maps, it is expected that the site is underlain by marine sediments of clay and silt. Interbedded limestone and shale of the Lindsay formation should be expected from about 25 to 50 metres below ground surface.

Fill material associated with previous development of the area should be expected near the existing structures on the south and west portions of the site.

3.0 SUBSURFACE INVESTIGATION

The field work for this investigation was carried out from April 2 to April 6, 2018. At that time, ten (10) boreholes, numbered 18-1 to 18-10, were advanced within the footprints of the proposed school buildings using a track mounted drill rig supplied and operated by George Downing Estate Drilling Limited of Grenville-Sur-La-Rouge, Quebec. Details of the boreholes are provided below:

- Four (4) boreholes (18-1 to 18-4) were advanced to depths between 9.8 and 10.4 metres below ground surface in the proposed footprint of the elementary school in the southwest quadrant of the site.

- Four (4) boreholes (18-5 to 18-8) were advanced to depths between 9.8 and 10.4 metres below ground surface in the proposed footprint of the secondary school in the northeast quadrant of the site.

- Two (2) boreholes (18-9 and 18-10) were advanced to a depth of 4.6 metres below ground surface on other areas of the site to facilitate environmental testing of the soil and groundwater. Results for the environmental testing will be provided in a separate report.

- A standpipe piezometer was installed in each borehole measure the stabilized groundwater levels.

Standard penetration tests were carried out in the boreholes, and samples of the soils encountered were recovered using a 50-millimetre diameter split barrel sampler. In-situ vane shear testing was carried out where possible in the boreholes to measure the undrained shear strength of the silty clay. Relatively undisturbed samples of the native deposits of silty clay were also obtained in boreholes 18-1 to 18-3 and in boreholes 18-5 to 18-7 at selected depth intervals with a Shelby tube sampler. Dynamic cone penetration testing (DCPT) was conducted in boreholes 18-4 and 18-6 to practical refusal. The samples were returned to our laboratory for examination by the project engineer. One (1) groundwater sample recovered from borehole 18-3 and one (1) groundwater sample from borehole 18-6 were sent for basic chemical testing relating to corrosion of buried concrete and steel.

The field work was supervised throughout by a member of our engineering staff who located the boreholes, logged the samples, and observed the in-situ testing. Following the field work, the soil samples were returned to our laboratory for examination by a geotechnical engineer. Select samples of the soil were tested for water content, grain size distribution, and Atterberg Limits testing.

The approximate locations of the boreholes are shown on the Borehole Location Plan, Figure 2. Descriptions of the subsurface conditions logged in the boreholes are provided on the Record of Borehole sheets in Appendix A. The results of the laboratory classification testing are provided on the Record of Borehole sheets and on Figures B1 to B2 in Appendix B. The results of the chemical analysis relating to corrosion of buried steel and concrete on the two (2) groundwater samples collected are provided in Appendix C.

The borehole locations were selected and determined relative to existing site features by GEMTEC personnel. Elevations were measured using our Trimble R10 GPS equipment and are referenced to geodetic datum.

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 exploration, the frequency and recovery of samples, the method of sampling, and the uniformity of the subsurface conditions. Subsurface conditions at other than the borehole locations may vary from the conditions encountered in the boreholes. 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 as part of this investigation.

4.2 Fill Material

Fill material was encountered from the surface at boreholes 18-1 to 18-4. The total thickness of the fill material is about 1.4 metres in boreholes 18-1, 18-2 and 18-4, and 0.6 metres in borehole 18-3. The fill material in boreholes 18-1 and 18-4 is composed of dark brown silty sand, with gravel and fragments of asphalt and other debris. The fill material in borehole 18-2 is composed of dark brown silty sand with trace clay and organic material. The fill material in borehole 18-3 is composed of grey brown silty clay with some sand and gravel.

The water content measured in the fill material ranges from about 12 to 18 percent.

4.3 Topsoil

In borehole 18-2, topsoil composed of dark brown sandy silt with organic material was encountered under the fill material at a depth of 1.3 metres below surface grade (elevation 88.7 metres, geodetic datum) and has a thickness of 0.3 metres.

4.4 Silty Clay

4.4.1 Silty Clay with Organic Material

A layer of brown silty clay with trace sand and organic material was encountered from the ground surface in boreholes 18-5 to 18-10. The thickness of this layer ranges from 0.1 to 0.8 metres.

Standard penetration tests (SPT) carried out in the layer of silty clay with organic material in boreholes 18-5 to 18-10 gave an N values ranging from 3 to 5 blows per 0.3 metres of penetration, which reflects a very stiff consistency.

The water content measured in a sample of the layer of silty clay with trace sand and organic material is about 40 percent.

4.4.2 Silty Clay Weathered Crust

A layer of weathered brown silty clay was encountered below the fill material in boreholes 18-1, 18-3 and 18-4, below the topsoil in borehole 18-2, and below the layer of silty clay with organic material in boreholes 18-5 to 18-8 at depths ranging between 0.1 and 1.6 metres below ground surface (elevation 86.1 to 87.2 metres, geodetic datum). SPT N values recorded within the weathered crust layer generally range from 2 to 12 blows per 0.3 metres of penetration, indicating a very stiff consistency, based on our local experience with clays in the Ottawa area.

The water content measured in the weathered crust layer ranges from about 28 to 59 percent.

4.4.3 Grey Silty Clay

The weathered crust transitions to grey silty clay at depths ranging between 2.3 and 3.8 metres below surface grade (elevation 84.0 to 85.1 metres, geodetic datum) at all borehole locations. All boreholes were terminated within the grey silty clay at depths ranging between 4.6 and 10.4 metres below surface grade.

SPT N values recorded within the grey silty clay generally range from static weight of hammer to 3 blows per 0.3 metres of penetration. The undrained shear strength measured in the grey silty clay ranges from 15 to 44 kilopascals, which corresponds to a soft to firm consistency. The corresponding remolded values range from 2 to 9 kilopascals. The ratio of the undrained shear strength to the remolded shear strength indicates that the sensitivity of the grey silty clay deposit is extra sensitive to medium sensitivity.

Representative samples of the silty clay were tested for:

- Moisture content;
- Grain size distribution; and,
- Atterberg limits.

The moisture content of the grey silty clay ranges from about 74 to 89 percent.

Two (2) grain size distribution tests were undertaken on samples of grey silty clay from boreholes 18-2 and 18-8. The results are provided on Figure B1 (Appendix B) and summarized in Table 4.1.

Table 4.1 – Summary of Grain Size Distribution Testing (Grey Silty Clay)

Location	Sample Number	Sample Depth (metres)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)
18-2	7	5.3 – 5.9	0	1	23	76
18-8	5	4.6 – 5.2	0	1	26	73

Four (4) Atterberg limits tests were undertaken on samples of the grey silty clay from boreholes 18-2, 18-3, 18-5 and 18-8. The results are provided on Figure B2 (Appendix B), on the Record of Borehole sheets (Appendix A) and are summarized in Table 4.2.

Table 4.2 – Summary of Atterberg Limits and Moisture Content Testing (Grey Silty Clay)

Location	Sample Number	Sample Depth (metres)	Moisture (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
18-2	7	5.3 – 5.9	86.6	58.6	22.8	35.8
18-3	7	6.1 – 6.7	87.6	55.5	25.1	30.4
18-5	6	4.6 – 5.2	81.5	56.5	22.7	33.8
18-8	5	4.6 – 5.2	85.3	55.4	25.4	30.0

As indicated on Figure B2, the grey silty clay has a high plasticity. It should be noted that the moisture contents of the grey silty clay samples are above respective the liquid limit values.

4.5 DCPT Results

As indicated on the Record of Borehole sheets (Appendix A), DCPTs were carried out in boreholes 18-4 and 18-6 between the depths of 10.7 and 36.5 metres below ground surface and 10.2 and 31.1 metres below ground surface, respectively. Based on the number of blows per 0.3

metres of penetration, possible glacial till was encountered in boreholes 18-4 and 18-6 at 35.1 and 32.0 metres below ground surface, respectively (elevation 52.8 metres and 54.8 metres, geodetic datum). It should be noted that the results of the dynamic cone penetration testing carried out in the above boreholes may have been influenced by friction acting along the length of rods.

Practical refusal to further advancement of the dynamic cone occurred in boreholes 18-4 and 18.6 at 36.5 and 32.1 metres below ground surface, respectively (elevations of 51.4 and 54.6 metres, geodetic datum) on the inferred bedrock surface. It should be noted that practical dynamic cone refusal can sometimes occur within cobbles and boulders and may not necessarily be representative of the upper surface of the bedrock.

4.6 Groundwater Conditions

The stabilized groundwater conditions were measured on April 9 and April 18, 2018 in the standpipe piezometers in each borehole. The groundwater levels are summarized in Table 4.3.

Table 4.3 – Groundwater Depth and Elevation

	April 9, 2018		April 18, 2018	
Borehole	Groundwater Depth Below Existing Ground Surface (metres)	Groundwater Elevation (metres, geodetic datum)	Groundwater Depth Below Existing Ground Surface (metres)	Groundwater Elevation (metres, geodetic datum)
18-1	1.6	86.0	0.9	86.7
18-2	1.4	86.6	0.9	87.1
18-3	2.0	85.4	0.2	87.2
18-4	1.2	86.6	0.8	87.1
18-5	3.3	83.5	1.2	85.6
18-6	0.3	80.4	0.1	86.6
18-7	6.4	86.7	2.3	84.8
18-8	4.8	82.1	1.9	84.9
18-9	0.7	86.6	0.1	87.2
18-10	0.5	86.9	0.3	87.1

It should be noted that groundwater levels will fluctuate seasonally and may be higher during wet periods of the year, such as the early spring or fall, or following periods of heavy precipitation.

4.7 Soil Chemistry Relating to Corrosion

The results of chemical testing on groundwater samples recovered from boreholes 18-3 and 18-6 are provided in Appendix C and are summarized in Table 4.4.

Table 4.4 – Summary of Corrosion Testing

Parameter	Borehole 18-3	Borehole 18-6
Chloride Content (mg/L)	3420	796
Conducitivty (uS/cm)	11,200	3,230
pH	7.8	8.0
Sulphate Content (mg/L)	303	79

5.0 GEOTECHNICAL DESIGN GUIDELINES

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 and are presented in a separate environmental report titled Phase Two Environmental Site Assessment, 2405 & 2419 Mer Bleue Road, Ottawa, Ontario and completed by GEMTEC.

5.2 Site Grade Raise Restrictions

This site is underlain by thick deposits of silty clay, which have a reduced capacity to support loads imposed by grade raise fill material, pavement structures and foundations for the buildings.

The placement of fill material on this site must therefore be carefully planned and controlled so that the stress imposed by the fill material does not result in excessive consolidation of the silty clay deposit. Concrete slabs, granular base materials, overall grade raise and pavement structures are considered grade raise filling. Groundwater lowering also results in a stress increase on the underlying silty clay deposit.

Based on the results of the subsurface investigation in conjunction with empirical calculations correlating the undrained shear strength of the soft to firm silty clay to the preconsolidation pressure, the maximum thickness of any grade raise filling should be limited to about 0.5 metres above the original ground surface at the site.

The preliminary grade raise restriction for this site has been calculated in order to limit the total settlement of the ground to about 25 millimetres in the long term. For design purposes, we have assumed that the groundwater lowering due to the development at this site will be at most 0.5 metres.

It is important to note that if excessive fill is placed across the site (i.e., in excess of 0.5 metres above the original grade at the site), the long-term settlement of the ground could be significant. For buildings supported on pile foundations, this could damage ground supported services entering the buildings and cause differential settlement between the structures and the finished grade around the proposed buildings.

It may be possible to improve the site grade raise restriction by carrying out laboratory oedometer consolidation testing on the thin walled Shelby tubes samples of the softer portion of the grey silty clay recovered during the geotechnical investigation. Further details could be provided upon request.

5.3 Foundation Design

Based on the subsurface conditions encountered and the anticipated building loads, it is our opinion that a deep foundation will most likely be required for this site. A shallow footing design could be considered as an alternative but will have a limited bearing capacity and footing size. The two following foundation options were considered for the proposed multi-storey buildings:

- Option 1: Conventional spread footing foundations; and,
- Option 2: End bearing piles on bedrock.

The above alternatives are discussed in greater detail below.

Option 1: Conventional Spread Footing Foundations

The bearing pressures for strip or pad footing foundations at this site are based on the necessity to limit the stress increase on the softer grey silty clay layer below the weathered crust to an acceptable level so that foundation settlements will not be excessive. Four important parameters in calculating the stress increase on the grey silty clay beneath the weathered crust are:

- The thickness of the soil beneath the base of the foundation and the surface of the grey silty clay;
- The size, type and loading of the foundation;
- The amount of surcharge (fill, etc.) in the vicinity of the foundation; and
- The amount of post-development groundwater lowering at the site.

From a spread footing design perspective, it is preferable to maximize the vertical separation between the underside of the footings and the surface of the softer, grey silty clay to distribute the foundation loads onto the softer, grey silty clay at depth. This can be achieved by founding the structures as high as practical within the soil profile and minimizing the amount of fill (surcharge) on the site.

For preliminary planning and design purposes, the parameters listed in Table 5.1 could be considered for the design of the foundations.

Table 5.1 – Bearing Capacity for Spread Footing Foundations

Type of Footing	Maximum Footing Depth Below Original Ground Surface (metres)	Maximum Size of Strip Footing (metres)	Net Geotechnical Reaction at Serviceability Limit State[1] (kilopascals)	Factored Net Geotechnical Resistance at Ultimate Limit State (kilopascals)
Spread Footing	1.0	1.0	50	150

Notes:
1. The total and differential settlement of the foundation at SLS should be less than 20 and 25 millimetres, respectively.

For the purpose of this assessment we have considered a long-term groundwater lowering at the site equal to 0.5 metres below the highest measured groundwater level. Provided that any loose or disturbed soil is removed from the bearing surfaces, the post construction total and differential settlement of the footings at SLS should be less than 25 and 20 millimetres. These settlements, and the bearing capacities provided in Table 5.1, assume that the fill materials are placed in accordance with the site grade raise restrictions.

Option 2: End Bearing Piles on Bedrock

In our opinion deep foundations will be required to support the anticipated loads of the proposed multi-storey buildings. Low displacement steel piles driven to bedrock are best suited for this site. Buoyancy (uplift) issues are anticipated if closed ended steel pipe piles are used since the silty clay will remold around the piles during installation of the piles. Therefore, consideration should be given to using steel H piles, which are not susceptible to buoyancy effects. Pile capacities at this site will depend on pile type, pile dimensions, pile material and bedrock conditions.

It should be noted that the top of bedrock was not confirmed at this site thus if the foundations are to be founded on piles we recommended coring bedrock at several locations to provide more information on the bedrock elevations across the sites.

We provide the following general pile recommendations:

- The pile cross section should be confirmed once structural loads are known. GEMTEC would be pleased to review pile types once the structural loads have been finalized.

- The pile driving contractor should be required to submit the pile design and pile driving criteria for review prior to pile driving at this site. An allowance should be made in the specifications for re-tapping all of the piles at least once after a minimum period of 24 hours to confirm the permanence of the pile set.

- A resistance factor of 0.4 should be applied to the Ultimate Limit State (ULS) resistance. If dynamic pile testing is undertaken, the resistance factor may be increased from 0.4 to 0.5. The ULS resistance will govern the design since the stresses required to induce Serviceability Limit State (SLS) criteria for piles terminated on bedrock will exceed those at ULS. Therefore, the SLS resistance has not been presented in this report.

- The refusal criteria will be highly dependent on the contractor's pile driving equipment. Typically, for the drop hammer type piling rigs available in Eastern Ontario, refusal criteria of 10 blows for the last 25 millimetres of penetration would be sufficient to achieve the above loads, assuming that a hammer with a rated energy of about 1,650 ft-lbs/in^2 or 350 Joules/cm^2. The actual hammer energy required to finalize the piles may vary depending on soil/bedrock conditions at each location and efficiency of the pile driving system.

- Driving criteria should be established using a Wave Equation Analysis once the hammer details are established.

- As a general contingency, we recommend that a minimum nominal corrosion of 1/16 inch (1.6 millimetres) be applied to the pile cross sectional area.

- In order to increase the resistance factor from 0.4 to 0.5, dynamic pile testing using a pile driving analyzer (PDA) should be undertaken on a minimum of 10 percent of the driven piles. We recommend that 24 hour re-strike testing be carried out on PDA tested piles to evaluate pile relaxation. Piles that relax should be driven and tested again 24 hours later.

- GEMTEC provides dynamic pile testing services using our pile driving analyzer and would be pleased to provide this service upon request.

- Full time inspection of pile driving by qualified geotechnical personnel is recommended.

- We would not expect any downdrag loading if the grade raise restriction limits are respected.

- The weathered and grey silty clay soils at this site are sensitive to disturbance from ponded water and construction traffic. If pile driving is carried out from the future subgrade level, allowance should be made to place a granular layer composed of at least 450 millimetres of OPSS Granular B Type II with a woven geotextile separator over the subgrade surface to provide a working mat for the pile driving and reduce the disturbance to the soil below the proposed buildings. Any disturbed soil should be removed from below the concrete slabs. Alternatively, excavation could be carried out after the pile driving and the subgrade surface covered with a 50 to 75 millimetre thick concrete mud mat.

5.4 Earth Pressures on Foundation Walls

In the event that retaining wall structures are required as part of the design, the following earth pressure parameters can be used for the retaining walls.

5.4.1 Active Earth Pressures

The static active thrust (P_a) acting on the wall should be calculated using the following formula:

$$P_a = 0.5\, K_a\, \gamma\, H^2$$

where;

- P_a: Static active thrust component (kilonewtons);
- γ: Moist material unit weight (kN/m^3);
- K_a: “Active” earth pressure coefficient;
- H: Wall height (metres).

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

$$P_{ae} = P_a + P_e$$

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

$P_e = 0.5 (K_{ae} - K_a) \gamma H^2$

where;

- P_e: Dynamic active thrust component (kilonewtons);
- γ: Moist material unit weight (kN/m^3);
- K_a: "Active" earth pressure coefficient;
- K_{ae}: Dynamic active earth pressure coefficient;
- H: Wall height (metres).

The static thrust component (P_a) acts at a point located H/3 above the base of the wall. During seismic shaking, the dynamic active 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 active thrust components acting on the wall.

Table 5.2 – Summary of Soil Parameters for Active Wall

Parameter	OPSS Granular B Type II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	38
"Active" Earth Pressure Coefficient, K_a, assuming horizontal backfill behind the structure	0.24
Dynamic Active Earth Pressure Coefficient, K_{ae}, assuming horizontal backfill behind the structure	0.35[1]

Notes:

1) According to the 2012 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 0.40 (Site Class E). The dynamic active earth pressure coefficient was calculated using the method suggested by Mononobe and Okabe, assuming a horizontal seismic coefficient, k_h, of 0.20 (one half the corrected PGA) and assuming that the vertical seismic coefficient, k_v, is zero.

5.4.2 At Rest Earth Pressures

The static at rest thrust (P_o) acting on the wall 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 5.3 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 II
Material Unit Weight, γ (kN/m^3)	22
Estimated Friction Angle (degrees)	38
"At Rest" Earth Pressure Coefficient, K_o, assuming horizontal backfill behind the structure	0.38
Dynamic At Rest Earth Pressure Coefficient, K_{oe}, assuming horizontal backfill behind the structure	0.52[1]

Notes:

1) According to the 2012 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 0.40 (Site Class E). 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 0.40 (taken as the corrected PGA) and assuming that the vertical seismic coefficient, k_v, is zero.

5.5 Frost Protection of Foundations

At least 1.5 metres of earth cover should be provided for frost protection purposes for footings, exterior pile caps and grade beams that are backfilled with well graded, non-frost susceptible sand or sand and gravel. Isolated, exterior footings, pile caps, or grade beams constructed in areas that are to be cleared of snow during the winter period should be provided with at least 1.8 metres of earth cover for frost protection purposes if they are backfilled with well graded, non-frost susceptible sand or sand and gravel. Where less than the required depth of soil cover can be provided, the pile caps, grade beams, and footings can be protected from frost by using a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

5.6 Seismic Design of Proposed Structures

Based on the vane shear strength data and the relatively thick soft to firm grey silty clay deposits, it is our opinion that Site Class E is appropriate for this specific site according to the 2015 National Building Code of Canada and the 2017 Ontario Building Code.

Based on the results of the investigation, there is no potential for liquefaction of the overburden deposits at this site.

5.7 Corrosion of Buried Concrete and Steel

The measured sulphate concentration in the groundwater recovered from boreholes 18-3 and 18-6 is 303 and 79 milligrams per litre, respectively. According to Canadian Standards Association

(CSA) “Concrete Materials and Methods of Concrete Construction”, the concentration of sulphate in the groundwater and soil can be classified as low to moderate. For low exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with General Use (formerly Type 10) cement. For moderate exposure conditions, any concrete that will be in contact with the native soil or groundwater should be batched with Moderate sulphate-resistant cement. The design of any concrete should take into consideration freeze thaw effects and the presence of chlorides.

Based on the conductivity and pH of the groundwater recovered from boreholes 18-3 and 18-6, the groundwater samples recovered can be classified as very aggressive and aggressive respectively towards unprotected steel. The manufacturer of any buried steel elements that will be in contact with the soil and groundwater should be consulted to determine the durability of the product used. It is noted that the corrosivity of the ground water could vary throughout the year due to the application sodium chloride for de-icing.

5.8 Recommended Additional Investigations

If pile foundations are being considered for the proposed buildings, it is recommended that additional boreholes be advanced to confirm the type, quality and depth of bedrock through coring.

As mentioned in Section 5.2, due to the soft clay encountered across the site, it is recommended that laboratory consolidation testing be carried out on the undisturbed samples of clay obtained in this investigation to potentially improve grade raise restrictions and allowable bearing values.

It may be possible to upgrade the seismic Site Class to a Site Class D, which may reduce overall design and construction costs, by determining the shear wave velocity at the site using Multichannel Analysis of Surface Waves (MASW) testing. However, it is noted that, based on our experience in the area, there is a low probability of improving the seismic Site Class if shear wave velocity testing is carried out.

6.0 ADDITIONAL CONSIDERATIONS

6.1 Excavation

The excavation for the proposed buildings will be carried out mostly through topsoil, fill materials, weathered silty clay and grey silty clay.

The sides of the excavations in overburden 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 and weathered, brown silty clay 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, extending from the bottom of the excavation.

It is our experience that the upper part of the weathered silty clay (i.e., within 0.3 to 0.5 metres from original ground surface) may be impacted by past frost action. During removal of the topsoil, the upper part of the silty clay could unavoidably peel upwards and become disturbed. Where this occurs in the proposed parking and access roadway areas, the upper part of the silty clay should be recompacted in place using suitable compaction equipment. Within the proposed buildings, it will likely be necessary to remove and replace the silty clay soil with imported granular material, such as OPSS Granular B Type II.

6.2 Slab on Grade

To provide predictable settlement performance of the slab on grade, all loose soil, fill, disturbed soil and organic soil should be removed from below the slab areas. The base for the floor slab should consist of at least 300 millimetres of OPSS Granular A.

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level. If any areas of the building are to remain unheated during the winter period, thermal protection of the materials beneath the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary.

Although feasible, a basement is not ideally suited for this site from a geotechnical point of view since a conventional drained basement could result in excessive groundwater lowering and possible additional ground settlement due to consolidation of the grey silty clay.

6.3 Access Roadways and Parking Areas for the Proposed Development

It is suggested that the access roadways and parking areas to be constructed using the following pavement structure:

- 80 millimetres of HL3 or Superpave 12.5 asphaltic concrete, over
- 150 millimetres of OPSS Granular A base, over
- 300 millimetres of OPSS Granular B Type II subbase.

Our pavement structure recommendation could be refined once traffic loads and volume are known.

For any access roadways which will be used by heavy trucks or fire trucks, the asphaltic concrete surfacing thickness should be increased to 100 millimetres and the thickness of the subbase layer increased to 450 millimetres.

The above pavement structure exceeds the site grade raise restrictions provided in Section 5.2. Therefore, subexcavation of underlying native soil is required to accommodate the pavement structure.

Additional details regarding subgrade preparation, grade raise restrictions on accessways and parking areas, compaction, drainage and effects of soil disturbance and construction traffic can be provided as the design progresses.

6.4 Seepage Barriers

To prevent the granular bedding in the services trench from acting as a "French Drain" and thereby resulting in groundwater lowering, seepage barriers should be installed along the service trenches just inside the property lines. 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.

6.5 Effects of Trees

This site is underlain by deposits of silty clay, a material which is known to be susceptible to shrinkage with a change/reduction in moisture content. Research by the Institute for Research in Construction (formerly the Division of Building Research) of the National Research Council of Canada has shown that trees can cause a reduction of moisture content in the silty clays in the Ottawa area, which can result in significant settlement/damage to nearby buildings supported on shallow foundations bearing on or above the silty clay. Therefore, no deciduous trees should be permitted closer to the buildings (or any ground supported structures which may be affected by settlement) than the ultimate height of the trees.

For groups of trees or trees in rows, the separation distance should be increased to 1.5 times the ultimate height of the trees.

The effects of existing and future trees on the proposed buildings, services and other ground supported structures should be considered in the landscaping design.

6.6 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, excavation, pile driving, 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. We recommend that preconstruction surveys be carried out on the adjacent structures to mitigate potential claims.

6.7 Winter Construction

In the event that construction is required during freezing temperatures, the soil below the proposed buildings should be protected immediately from freezing using straw, propane heaters and insulated tarpaulins, or other suitable means.

Any open excavations should be opened for as short a time as practicable. The materials on the sides of the excavation 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.

Provision must be made to prevent freezing of any soil below the level of any existing structures or services. Freezing of the soil could result in damage to structures or services.

6.8 Design Review and Construction Observation

The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the final 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 building and access roadways 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.

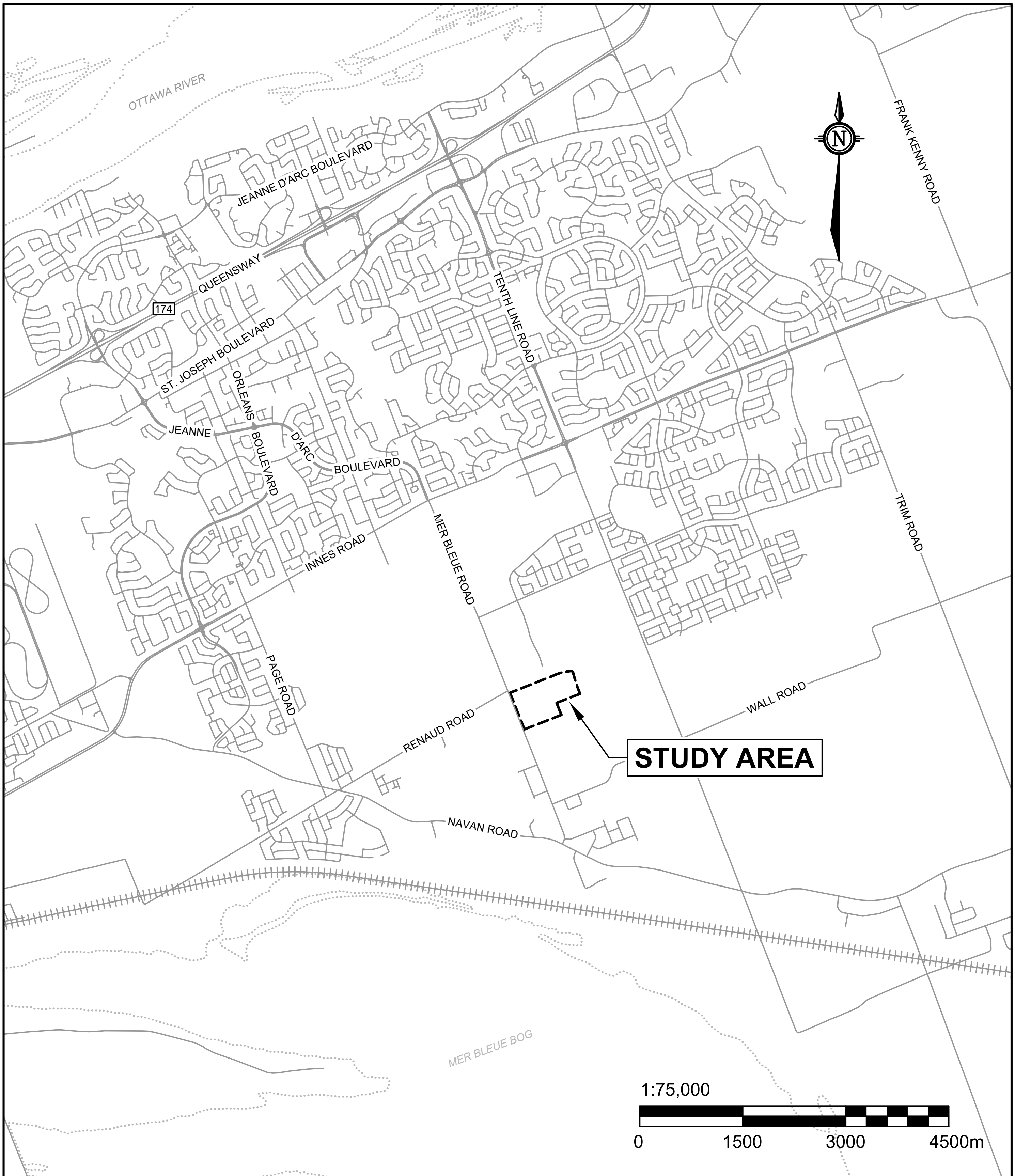
Full time field review will be required for pile foundations to check that any piling installation meets specifications. The pile type, installation procedures and refusal criteria proposed by the piling contractor should be reviewed and accepted by the geotechnical engineer prior to the start of construction. Copies of mill certificates for the piling material should also be submitted and accepted before delivery of the material to the site.

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.

Kelsey Holkestad, B.Eng., EIT

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

LICENSED PROFESSIONAL ENGINEER
J. A. CHOLEWA
100119055
03 May 2018
PROVINCE OF ONTARIO



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CONSULTING ENGINEERS AND SCIENTISTS

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

Project				Drawing	
GEOTECHNICAL & PHASE II ESA MER-BLEUE SCHOOL 2401 & 2419 MER-BLEUE ROAD OTTAWA, ONTARIO				KEY PLAN	
Drwn By	Chkd By	Date	Project No.	Revision No.	
S.L.	N.S.	APRIL 2018	62721.07	0	FIGURE 1

LEGEND

BH18-1 87.61 — BOREHOLE LOCATION (Environmental/ Geotechnical)

GROUND SURFACE ELEVATION IN METRES GEODETIC DATUM

BH18-9 87.31 — BOREHOLE LOCATION (Environmental)

STUDY AREA

Scale

1:1250

0 25 50 75m

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CONSULTING ENGINEERS AND SCIENTISTS

32 Steacie Drive
Ottawa, ON K2K 2A9
Tel: (613) 836-1422
www.gemtec.ca
ottawa@gemtec.ca

Drawing

BOREHOLE LOCATION PLAN

Client

CONSEIL DES ÉCOLES PUBLIQUES DE L'EST DE L'ONTARIO

Project		GEOTECHNICAL & PHASE II ESA MER-BLEUE SCHOOL 2401 & 2419 MER-BLEUE ROAD OTTAWA, ONTARIO
62721.07		
Drwn by	Chkd by	
S.L.	K.R.	

Date	Rev.	
APRIL 2018	0	**FIGURE 2**

APPENDIX A

Record of Borehole Sheets
List of Abbreviations and Terminology

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS auger sample
CA casing sample
CS chunk sample
BS Borros piston sample
GS grab sample
DO drive open
MS manual sample
RC rock core
ST slotted tube
TO thin-walled open Shelby tube
TP thin-walled piston Shelby tube
WS wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimetre required to drive a 50 mm drive open sampler for a distance of 300 mm. 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 hammer dropped 760 mm to drive a 50 mm diameter, 60º cone attached to 'A' size drill rods for a distance of 300 mm.

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.

SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH sieve and hydrometer analysis
U unconfined compression test
Q undrained triaxial test
V field vane, undisturbed and remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

Consistency	'N' Value
Stiff to Very Stiff	≥ 2

The consistency of unweathered, grey clay should only be based on the undrained shear strength.

LIST OF COMMON SYMBOLS

c_u undrained shear strength
e void ratio
C_c compression index
c_v coefficient of consolidation
k coefficient of permeability
I_p plasticity index
n porosity
u pore pressure
w moisture content
w_L liquid limit
w_P plastic limit
ϕ^1 effective angle of friction
γ unit weight of soil
γ^1 unit weight of submerged soil
σ normal stress

RECORD OF BOREHOLE 18-1

PROJECT: 62721.07

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: April 3, 2018

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

Boring method: Power Auger, 200 mm Diameter Hollow Stem

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m
0	Ground Surface	87.61			
	Dark brown silty sand, gravel, fragments of asphalt and other debris (FILL MATERIAL)		1	50 DO	29
		86.24 / 1.37	2	50 DO	32
	Very stiff, brown SILTY CLAY (weathered crust)		3	50 DO	12
			4	50 DO	5
		83.95 / 3.66	5	50 DO	5
	Firm to soft, grey SILTY CLAY		6	50 DO	2
			7	TW	
	End of Borehole	77.85 / 9.76			

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80)

SHEAR STRENGTH Cu, kPa — nat. V - + Q - ● ; rem. V - ⊕ U - ○ (20, 40, 60, 80)

HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5}, 10^{-4}, 10^{-3}, 10^{-2})

WATER CONTENT, PERCENT — Wp, W, Wl (20, 40, 60, 80)

ADDITIONAL LAB. TESTING

PIEZOMETER OR STANDPIPE INSTALLATION:
Soil cuttings
Bentonite seal
Filter sand
22 mm diameter, 0.6 metres long well screen
Soil cuttings

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	1.60 ▽	86.01
18/04/18	0.90 ▼	86.71

DEPTH SCALE

1 to 100

GEMTEC

LOGGED: M.L.

CHECKED:

RECORD OF BOREHOLE 18-2

PROJECT: 62721.07

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: April 3, 2018

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Power Auger 200 mm Diameter Hollow Stem	Ground Surface	88.03					
		Dark brown silty sand, trace clay with organics (FILL MATERIAL)		1	50 DO	5		Soil cuttings
1			86.66 1.37	2	50 DO	8		
		Dark brown sandy silt with organic material (TOPSOIL)	86.41 1.62					
2		Very stiff, brown SILTY CLAY (Weathered Crust)		3	50 DO	9		
				4	50 DO	4		
3				5	50 DO	3		Bentonite seal
4		Firm to soft, grey SILTY CLAY	84.22 3.81	6	50 DO	2		Filter sand; 22 mm diameter, 0.6 metres long well screen
5–6				7	50 DO	WH for 0.3 metres	MH, See Fig. B1	Soil cuttings
8				8	50 DO	WH for 0.3 metres		
9				9	TW			
10		End of Borehole	77.66 10.37					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ ; 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5} 10^{-4} 10^{-3} 10^{-2}

WATER CONTENT, PERCENT: Wp W Wl ; 20 40 60 80

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	1.40 ▽	86.63
18/04/18	0.92 ▼	87.11

DEPTH SCALE

1 to 100

GEMTEC

LOGGED: M.L.

CHECKED:

RECORD OF BOREHOLE 18-3

PROJECT: 62721.07
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: April 3, 2018

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

Boring method: Power Auger, 200 mm Diameter Hollow Stem

Depth (m)	Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m	Additional Lab. Testing	Piezometer or Standpipe Installation
0	Ground Surface	87.35					
	Grey brown silty clay, some sand and gravel (FILL MATERIAL)	86.74 / 0.61	1	50 DO	15		Protective flushmount casing; Bentonite seal
1	Very stiff, brown SILTY CLAY (Weathered Crust)		2	50 DO	12		
2			3	50 DO	3	PHCs, VOCs, PAHs, and metals and inorganics + Duplicate	
			4	50 DO	2		Filter sand
3	Firm to soft, grey SILTY CLAY	84.30 / 3.05	5	50 DO	WH for 0.3 metres		50 mm diameter, 3.05 metres long well screen
5			6	TW			
			7	50 DO	WH for 0.3 metres		
6			8	50 DO	WH for 0.3 metres		Soil cuttings
9			9	50 DO	WH for 0.3 metres		
10	End of Borehole	76.98 / 10.37					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80
SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ : 20, 40, 60, 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}
WATER CONTENT, PERCENT: Wp W Wl : 20, 40, 60, 80

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	2.00 ▽	85.35
18/04/18	0.15 ▼	87.20

DEPTH SCALE
1 to 100

GEMTEC

LOGGED: M.L.
CHECKED:

RECORD OF BOREHOLE 18-4

PROJECT: 62721.07
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: April 2, 2018

SHEET 1 OF 2
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	87.84					Protective flushmount casing
0–1.37	Dark brown silty sand, gravel, fragments of asphalt and other debris (FILL MATERIAL)	86.47 / 1.37	1	50 DO	25	PAHs, VOCs, PAHs, and metals and inorganics	Bentonite seal
1.37–3.05	Very stiff, brown SILTY CLAY (Weathered Crust)	84.79 / 3.05	2	50 DO	12		
			3	50 DO	4		
3.05–10.37	Firm to soft, grey SILTY CLAY	77.47 / 10.37	4	50 DO	1		Filter sand
			5	50 DO	WH for 0.3 metres		50 mm diameter, 3.05 metres long well screen
			6	50 DO	WH for 0.3 metres		
			7	50 DO	WH for 0.3 metres		Soil cuttings
10.37–20	Possible clayey deposits						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ : 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5} 10^{-4} 10^{-3} 10^{-2}
WATER CONTENT, PERCENT: Wp W Wl: 20 40 60 80

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	1.20 ∇	86.64
18/04/18	0.76 ▼	87.08

DEPTH SCALE
1 to 100

GEMTEC

LOGGED: M.L.
CHECKED:

RECORD OF BOREHOLE 18-4

PROJECT: 62721.07

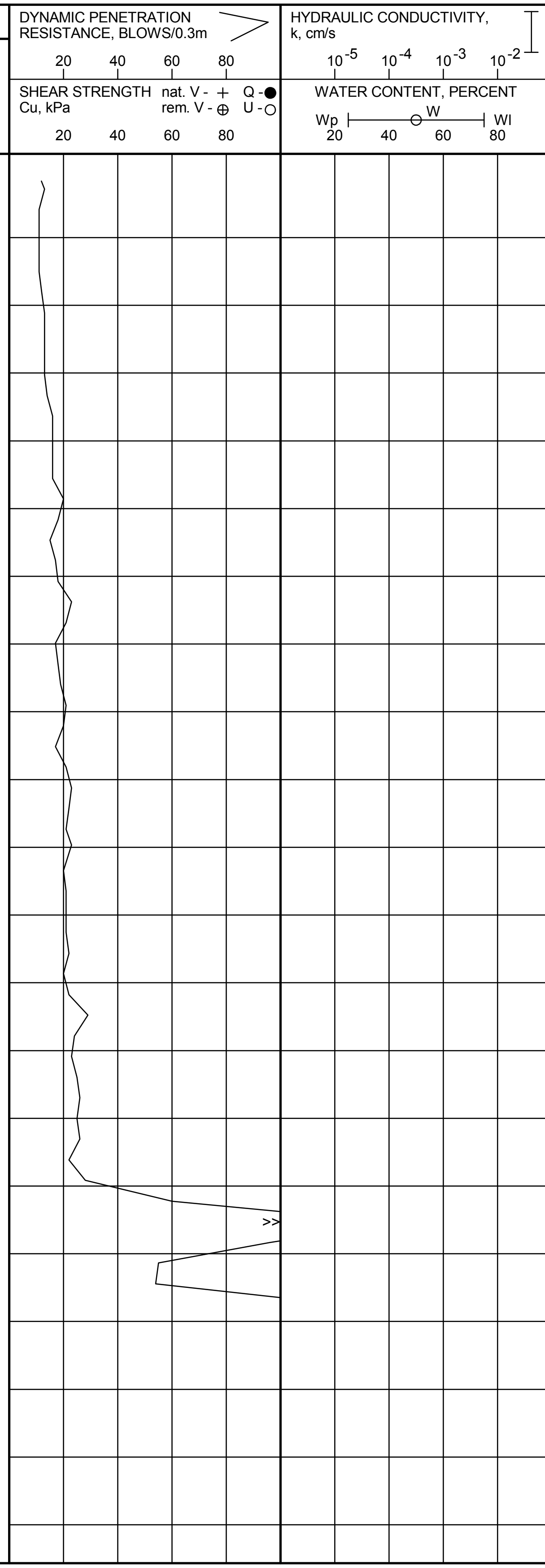
LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: April 2, 2018

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m / SHEAR STRENGTH Cu, kPa	HYDRAULIC CONDUCTIVITY, k, cm/s / WATER CONTENT, PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20–35	Dynamic Cone Penetration Test, 38 mm Diameter Cone										
35.06		Possible glacial till		52.78 / 35.06				>>			
36.48		DCPT Refusal. End of Borehole		51.36 / 36.48							



DEPTH SCALE

1 to 100

GEMTEC

LOGGED: M.L.

CHECKED:

PROJECT: 62721.07

RECORD OF BOREHOLE 18-5

SHEET 1 OF 1

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: April 6, 2018

SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES | BORING METHOD | SOIL PROFILE | SAMPLES | DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20, 40, 60, 80) | SHEAR STRENGTH Cu, kPa nat. V - + Q - ● rem. V - ⊕ U - ○ (20, 40, 60, 80) | HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}) | WATER CONTENT, PERCENT Wp ⊢ ○W ⊣ Wl (20, 40, 60, 80) | ADDITIONAL LAB. TESTING | PIEZOMETER OR STANDPIPE INSTALLATION

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	86.84
Brown SILTY CLAY, trace sand with organics	86.23 / 0.61
Very stiff, brown SILTY CLAY Weathered Crust)	84.10 / 2.74
Firm to soft, grey SILTY CLAY	76.47 / 10.37
End of Borehole	

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	4
2	50 DO	6
3	50 DO	4
4	50 DO	2
5	50 DO	WH for 300 mm
6	50 DO	WH for 300 mm
7	50 DO	WH for 300 mm
8	TW	

ADDITIONAL LAB. TESTING: PAHs, metals and inorganics, and OC pesticides PHCs, and BTEX

PIEZOMETER OR STANDPIPE INSTALLATION: Soil cuttings; Bentonite seal; Filter sand; 22 mm diameter, 0.6 metres long well screen; Soil cuttings

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	3.30 ▽	83.54
18/04/18	1.21 ▼	85.63

DEPTH SCALE

1 to 100

GEMTEC

LOGGED: M.L.

CHECKED:

PROJECT: 62721.07

RECORD OF BOREHOLE 18-6

SHEET 1 OF 2

LOCATION: See Borehole Location Plan, Figure 2

DATUM: Geodetic

BORING DATE: April 6, 2018

SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	86.77
Brown SILTY CLAY, trace sand with organics	86.16 / 0.61
Very stiff, brown SILTY CLAY (Weathered Crust)	84.48 / 2.29
Firm to soft, grey SILTY CLAY	77.02 / 9.75
Possible clayey deposits	

SAMPLE NUMBER	TYPE	BLOWS/0.3m
1	50 DO	3
2	50 DO	7
3	50 DO	5
4	50 DO	WH for 300 mm
5	50 DO	WH for 300 mm
6	50 DO	WH for 300 mm
7	TW	

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5} 10^{-4} 10^{-3} 10^{-2}

WATER CONTENT, PERCENT: Wp W Wl : 20 40 60 80

ADDITIONAL LAB. TESTING: PAHs, metals and inorganics, and OC Pesticides

PIEZOMETER OR STANDPIPE INSTALLATION: Bentonite seal; Filter sand; 50 mm diameter, 3.05 metres long well screen; Soil cuttings

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	6.40 ∇	80.37
18/04/18	0.14 ▼	86.63

DEPTH SCALE 1 to 100

GEMTEC

LOGGED: M.L.

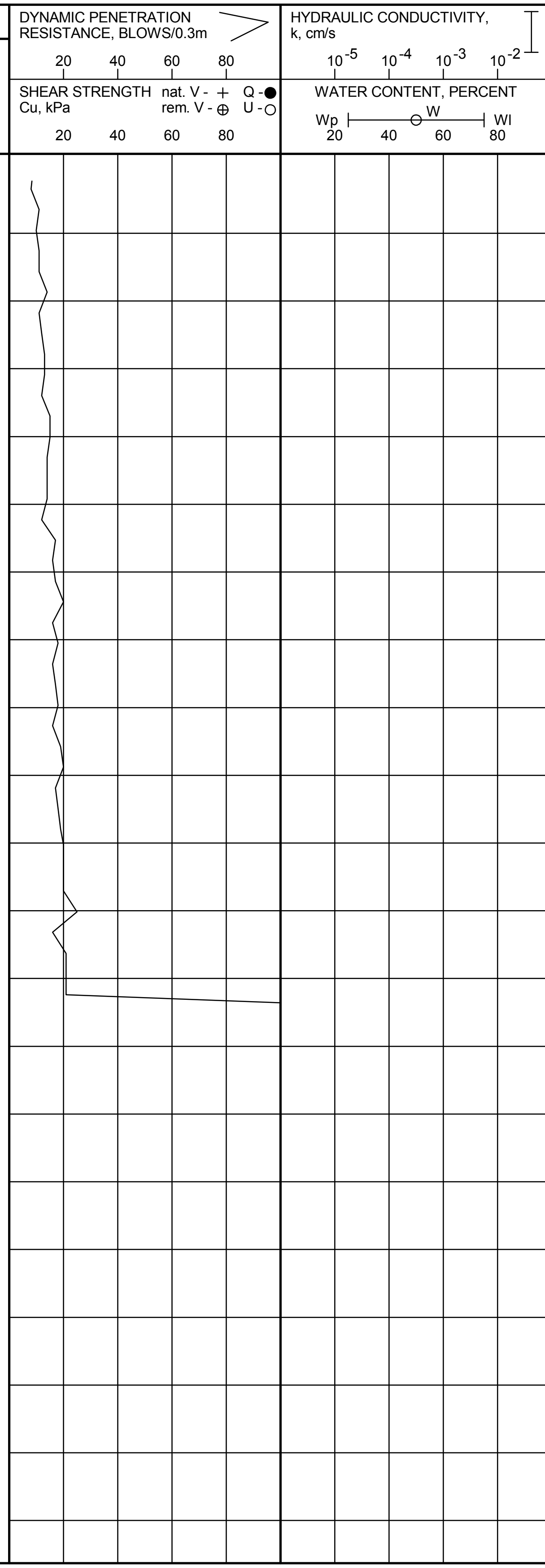
CHECKED:

RECORD OF BOREHOLE 18-6

PROJECT: 62721.07
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: April 6, 2018

SHEET 2 OF 2
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m; SHEAR STRENGTH Cu, kPa	HYDRAULIC CONDUCTIVITY, k, cm/s; WATER CONTENT, PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
20–32	Dynamic Cone Penetration Test, 38 mm Diameter Cone										
32				54.76							
		Possible glacial till		32.01 / 54.04							
		DCPT Refusal		32.13							
		End of Borehole									



DEPTH SCALE
1 to 100

GEMTEC

LOGGED: M.L.
CHECKED:

RECORD OF BOREHOLE 18-7

PROJECT: 62721.07

LOCATION: See Borehole Location Plan, Figure 2

BORING DATE: April 4, 2018

SHEET 1 OF 1

DATUM: Geodetic

SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

ELEV. / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
87.10	Ground Surface				
86.49 / 0.61	Brown SILTY CLAY, trace sand with organics	1	50 DO	5	PAHs, metalsn and inorgnaics, and OC pesticides
84.81 / 2.29	Very stiff, brown SILTY CLAY (Weathered Crust)	2	50 DO	4	
		3	50 DO	4	
76.73 / 10.37	Firm to soft, grey SILTY CLAY	4	50 DO	WH for 300 mm	
		5	TW		
		6	50 DO	WH for 300 mm	
		7	50 DO	WH for 300 mm	
	End of Borehole				

PIEZOMETER OR STANDPIPE INSTALLATION: Soil cuttings; Bentonite Seal; Filter sand; 22 mm diameter, 0.6 metres long well screen

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	0.40 ∇	86.70
18/04/18	2.32 ▼	84.78

DEPTH SCALE 1 to 100

GEMTEC

LOGGED: M.L.

CHECKED:

RECORD OF BOREHOLE 18-8

PROJECT: 62721.07
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: April 4, 2018

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES | BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

SOIL PROFILE

DESCRIPTION	ELEV. / DEPTH (m)
Ground Surface	86.87
Brown SILTY CLAY, trace sand with organics	86.11 / 0.76
Very stiff, brown SILTY CLAY (Weathered Crust)	84.58 / 2.29
Firm to soft, grey SILTY CLAY	76.81 / 10.06
End of Borehole	

SAMPLES

NUMBER	TYPE	BLOWS/0.3m
1	50 DO	2
2	50 DO	6
3	50 DO	WH for 300 mm
4	50 DO	WH for 300 mm
5	50 DO	WH for 300 mm
6	50 DO	WH for 300 mm

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● rem. V - ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5} 10^{-4} 10^{-3} 10^{-2}
WATER CONTENT, PERCENT: Wp W Wl : 20 40 60 80

ADDITIONAL LAB. TESTING: PAHs, and metals and inorganics PHCs, and BTEX; MH, See Fig. B1

PIEZOMETER OR STANDPIPE INSTALLATION: Soil cuttings; Bentonite Seal; Filter sand; 22 mm diameter, 0.6 metres long well screen

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	4.80 ▽	82.07
18/04/18	1.94 ▼	84.93

DEPTH SCALE
1 to 100

GEMTEC

LOGGED: M.L.
CHECKED:

RECORD OF BOREHOLE 18-9

PROJECT: 62721.07
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: April 2, 2018

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

BORING METHOD: Power Auger, 200 mm Diameter Hollow Stem

DEPTH SCALE METRES	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
0	Ground Surface	87.31					Bentonite seal
	Brown SILTY CLAY, trace sand with organics	87.21 / 0.10	1	50 DO	6	PAHs, metals and inorganics, and OC pesticides, PHCs, and BTEX	
1	Very stiff, brown SILTY CLAY (Weathered Crust)		2	50 DO	8		Filter sand
2			3	50 DO	4		50 mm diameter, 3.05 metres long well screen
			4	50 DO	2		
3	Firm to soft, grey SILTY CLAY	84.26 / 3.05	5	50 DO	WH for 300 mm		
4			6	50 DO	2		
	End of Borehole	82.74 / 4.57					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80
SHEAR STRENGTH Cu, kPa: nat. V - + ; rem. V - ⊕ ; Q - ● ; U - ○ : 20, 40, 60, 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}
WATER CONTENT, PERCENT: Wp, W, Wl: 20, 40, 60, 80

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	0.70 ▽	86.61
18/04/18	0.08 ▼	87.23

DEPTH SCALE
1 to 100

GEMTEC

LOGGED: M.L.
CHECKED:

RECORD OF BOREHOLE 18-10

PROJECT: 62721.07
LOCATION: See Borehole Location Plan, Figure 2
BORING DATE: April 2, 2018

SHEET 1 OF 1
DATUM: Geodetic
SPT HAMMER: 63.5 kg; drop 0.76 metres

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE – DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES – NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		Ground Surface		87.41					
0	Power Auger 200 mm Diameter Hollow Stem	Brown SILTY CLAY, trace sand with organics		87.31 / 0.10	1	50 DO	4	PHCs, VOCs, PAHs, and metals and inorganics	Bentonite seal
1		Very stiff, brown SILTY CLAY (Weathered Crust)			2	50 DO	9		Filter sand
2					3	50 DO	4		50 mm diameter, 3.05 metres long well screen
		Firm to soft, grey SILTY CLAY		85.12 / 2.29	4	50 DO	3		
3					5	50 DO	WH for 300 mm	PHCs, and VOCs	
4					6	50 DO	WH for 300 mm		
		End of Borehole		82.84 / 4.57					

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20, 40, 60, 80
SHEAR STRENGTH Cu, kPa: nat. V - + Q - ● ; rem. V - ⊕ U - ○ ; 20, 40, 60, 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-5}, 10^{-4}, 10^{-3}, 10^{-2}
WATER CONTENT, PERCENT: Wp — W — Wl ; 20, 40, 60, 80

GROUNDWATER OBSERVATIONS

DATE	DEPTH (m)	ELEV. (m)
18/04/09	0.50 ∇	86.91
18/04/18	0.31 ▼	87.10

DEPTH SCALE
1 to 100

GEMTEC

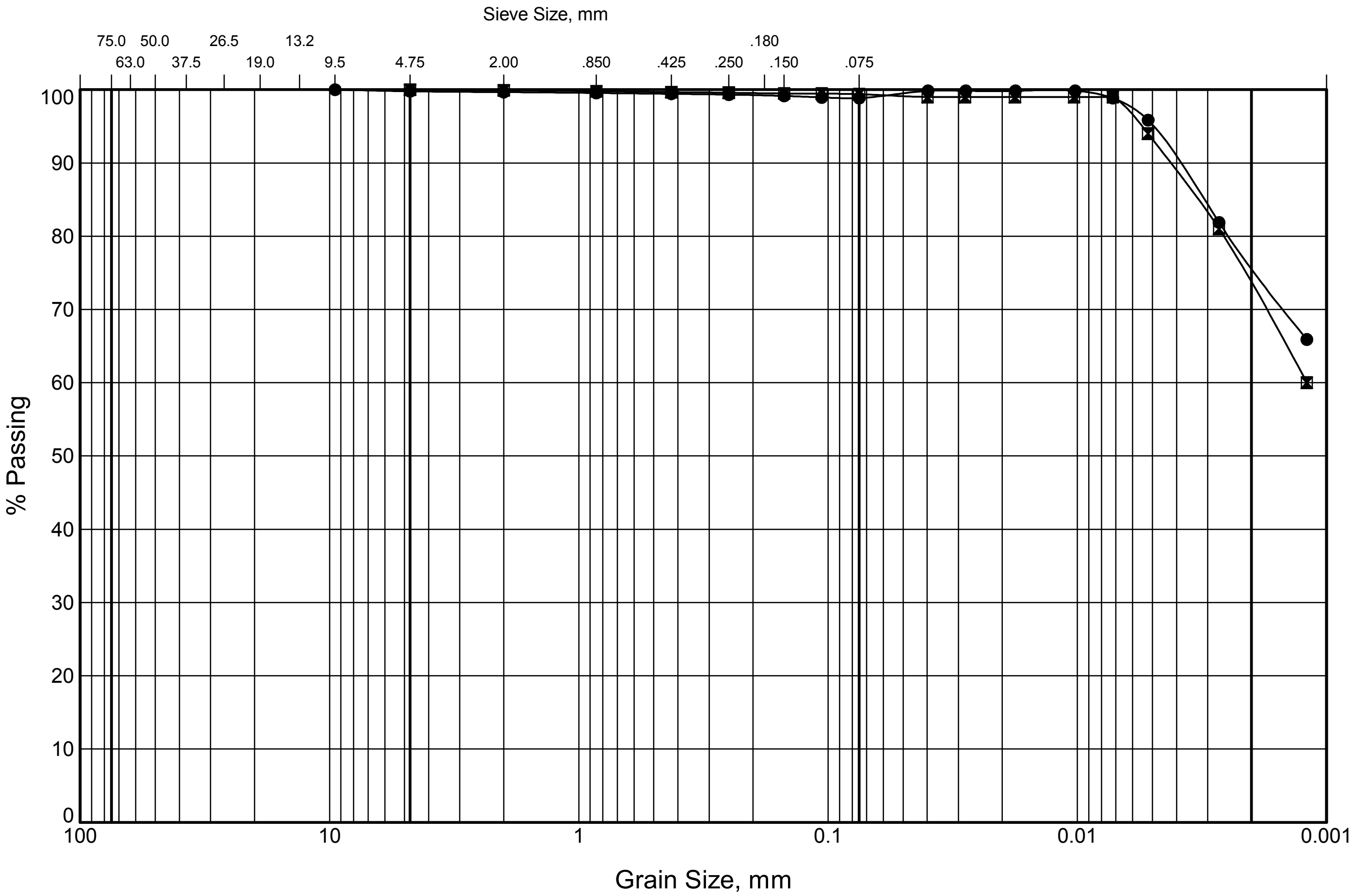
LOGGED: M.L.
CHECKED:

APPENDIX B

Results of Laboratory Testing
Figures B1 and B2

GRAIN SIZE DISTRIBUTION

FIGURE B1



COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
	GRAVEL		SAND				

Legend	Borehole	Sample	Depth (m)	% Gravel	% Sand	% Silt	% Clay
●	18-2	7	5.3 - 5.9	0	1	23	76
⧗	18-8	5	4.6 - 5.2	0	1	26	73

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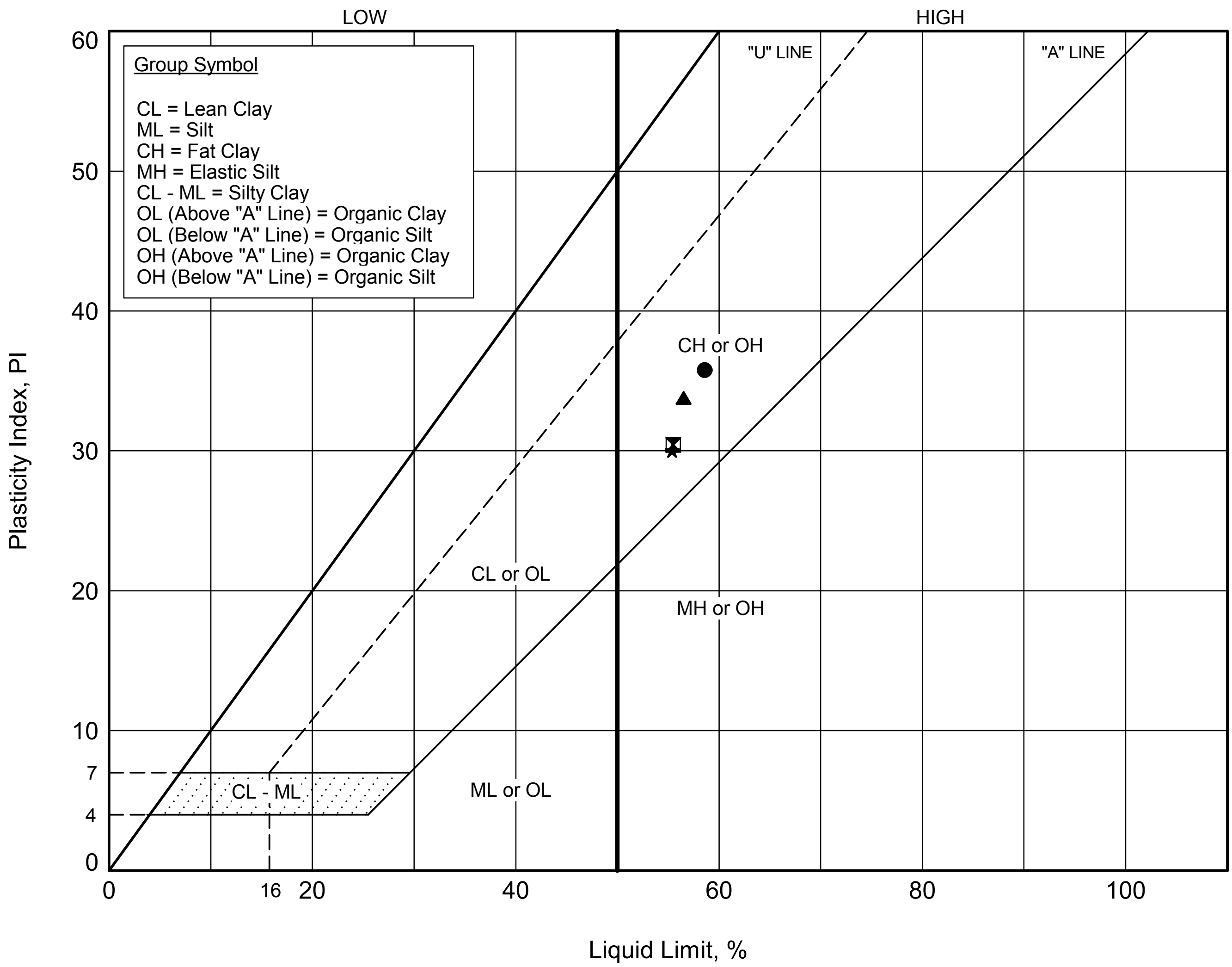
Date: April 2018

Project: 62721.07

PLASTICITY CHART	FIGURE B2



Legend	Borehole	Sample	Depth (m)	Water Content %	LL %	PL %	PI %
●	18-2	7	5.3 - 5.9	86.6	58.6	22.8	35.8
⧗	18-3	7	6.1 - 6.7	87.6	55.5	25.1	30.4
▲	18-5	6	4.6 - 5.2	81.5	56.5	22.7	33.8
★	18-8	5	4.6 - 5.2	85.3	55.4	25.4	30.0

HCE ATTERBERG LIMITS 62721.07 GINT LOGS APRIL 2018.GPJ HOULE CHEVRIER FEB 9 2011.GDT 20/4/18

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Date: April 2018
Project: 62721.07

APPENDIX C

Chemical Analyses of Groundwater
Samples Relating to Corrosion
Order No. 1815072

PARACEL
LABORATORIES LTD.

Order #: 1815072

Certificate of Analysis

Client: GEMTEC Consulting Engineers and Scientists Limited

Client PO:

Report Date: 12-Apr-2018

Order Date: 9-Apr-2018

Project Description: 62721.07

	Client ID:	BH18 - 3 Geo	BH18 - 6 Geo	-	-
	Sample Date:	04/09/2018 09:00	04/09/2018 09:00	-	-
	Sample ID:	1815072-01	1815072-02	-	-
	MDL/Units	Water	Water	-	-
General Inorganics					
Conductivity	5 uS/cm	11200	3230	-	-
pH	0.1 pH Units	7.8	8.0	-	-
Anions					
Chloride	1 mg/L	3420	796	-	-
Sulphate	1 mg/L	303	79	-	-

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environmental

field services

materials testing

civil

géotechnique

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