National Capital Commission

Soil Investigation
Trail #50 Bridge Replacement

Preliminary Report

Date: January 2018
Ref. N°: 033-B-0018588-1-GE-R-0001-0A

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Prepared by: 2018-01-30

Tommy Lampron, Eng.
Discipline Manager – Geotechnical
OIQ Member #5029258

Approved by : Yaya Coulibaly ing

Yaya Coulibaly, Eng., P.Eng., M.Eng.
Project Manager
OIQ Member #140220

900, de la Carrière Boulevard, Suite 100, Gatineau (Quebec) Canada J8Y 6T5 – 819.778.3143 | F 819.770.1373 – gatineau@englobecorp.com

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INTRODUCTION

Englobe Corp. services were selected by the **National Capital Commission (NCC)** to carry out an additional geotechnical study for the replacement of the Trail #50 pedestrian bridge in the Gatineau Park in Wakefield, Quebec.

The information collected during this investigation allowed us to determine the properties of subsoil materials and groundwater conditions to propose suitable recommendations for geotechnical project design, in regards to:

- Recommendations for temporary excavation and shoring;
- Temporary groundwater control;
- Frost protection;
- Bearing capacity for wooded caisson abutments;
- Seismic data;
- Recommendations for the repairs of the trail.

The terms governing this mandate are based on the statement contained in our proposal, identified 2017-P033-0368 and accepted by the client. This report contains a site description and investigation methods, as well as a detailed description of the nature and properties of subsoils materials encountered in the sounding and the conditions of groundwater. Finally, a section is devoted to the discussion of the results and geotechnical recommendations for the design of the project.

The terms defining the impact of the study are described in Appendix 1 of the report.

1 SITE AND PROJECT DESCRIPTION

The purpose of this project is the replacement of the Trail #50 pedestrian bridge, located approximately 190 meters west of Parking P50, on Pine Road, in Wakefield, Quebec. The bridge, built in 1992, is about 7 meters long and allow the crossing of Meech Creek. The bridge is founded on wood crib foundations.

Following October 2017 heavy rains, the bridge and the trail have suffered significant damage caused by runoff water.

The project consist of the reconstruction of the bridge and trail. The bridge will be expended by about eight (8) meters further on each abutment to prevent encroachment below the high water line.

Figure 1 shows an aerial view of the site under study.

Figure 1: Aerial view of the pedestrian bridge (Source: NCC)

2 INVESTIGATION PROCEDURES (FIELDWORKS)

2.1 LOCATION OF THE BOREHOLES

A site survey to determine the boreholes locations was carried out by Englobe representatives.

Borehole locations were surveyed by measuring the distance from an existing structure, shown on the as-built plan transmitted by the NCC (Ref.: Project # 17059, surveyed on October 24, 2017, and prepared on November 21, 2017).

The borehole elevations were taken using nail set horizontally in a tree located north-east from the bridge, with a reported elevation of 120.09 m taken from the as-built plan.

The location of the boreholes are shown on the plan 033-B-0018588-1-GE-D-0001-00 in the Appendix 4.

2.2 FIELDWORKS

The field investigation was performed on November 27th, 2017. It consisted of drilling two (2) boreholes, numbered BH-01-17 and BH-02-17. These boreholes were done using a "TM-3" drill located on the northern abutment (BH-01-17) and further on the trail (BH-02-17). Table 1 presents a summary of the boreholes.

Table 1: Boreholes Summary

Boreholes	Tests	Depth (M)
BH-01-17	Geotechnical samples, Standard penetration test (SPT), Field vane testing, Dynamic cone penetration test (DCPT), piezometer installation	28.50*
BH-02-17	Geotechnical samples, Standard penetration test (SPT)	3.05

**: End drilling after obtaining a refusal on a block, very dense soil or probable rock.*

The soil sampling was performed with a standard split spoon sampler of 51 mm outer diameter. The average penetration value N was also measured in accordance with the ASTM D-1586 requirements.

The dynamic cone penetration test was done for in borehole BH-01-17 to determine depth of dense soils, blocks or probable bedrock.

One field vane profile was performed in borehole BH-01-17 using a "Nilcon Vane". These tests were performed on an undisturbed soils at 1 m intervals. These tests allowed to measure the undrained shear strength of the undisturbed clay (Cu).

All the field work was done under the supervision of a full time experienced geotechnical technician from Englobe. The description of the different material layers met in the boreholes can be found on the individual boreholes reports in the Appendix 2.

2.3 LABORATORY WORK

All samples collected in the sounding were taken to our laboratory for analysis, identification and classification purposes. They have all been subject to a careful visual examination by a geotechnical engineer.

The soil samples collected from different depths have been submitted to laboratory testing in order to complete the information of the geotechnical characteristics gathered during field work. These tests were performed according to the requirements of the BNQ standards and the results are presented in the Appendix 3.

The following Table 2 presents the laboratory program.

Table 2: Laboratory testing program

Tests	Number
Sieve Analysis	3
Sedimentation Analysis	1
Atterberg Limits	3
Natural Water Content	8

All the samples collected in the boreholes, including those who were not subjected to laboratory tests, will be preserved for a period of six (6) months and this from the date of completion of field work. They will be destroyed unless a written notice as to their destination is transmitted to us.

3 NATURE AND PROERTIES OF SUBSOIL

This section discusses the stratigraphic units identified at the borehole locations on the studied site. Table 3 shows the distribution of stratigraphic units.

Table 3: Summary of stratigraphic units encountered in the boreholes

Borehole #	Granular fill (m)	Sand deposit (m)	Clay deposit (m)	Dense deposit (m)	End of borehole (m)
Bh-01-17	0.00 – 1.22	1.22 – 4.04	4.04 – > 8.23	At 25 m	28.50*
Bh-02-17	0.00 – 1.22	N/e	1.22 – > 3.05	N/e	3.05

**: End drilling after obtaining a refusal on a block, very dense soil or probable rock*

N/E : Not encountered.

3.1 GRANULAR FILL

From the ground surface, a granular fill was intercepted down to a depth of 1.22 m in both boreholes. This fill was followed by a sand deposit at the location of borehole BH-01-17, and by a clay deposit at the location of BH-02-17. A low amount of soil sample recoveries were obtained in borehole BH-01-17.

One (1) particle size analyse and one (1) determination of the natural moisture test were performed on the sample taken from this unit. The results are summarized in Table 4.

Table 4: Results of the sieves analyze on target samples - West Abutment

Boreholes #	Depth (m)	Gravel > 5 mm (%)	Sand < 5mm and > 80µm (%)	Silt and clay < 80µm (%)
BH-02-17	0.00 – 0.61	34.3	53.3	12.4

3.2 SAND DEPOSIT

At the location of the borehole BH-01-17, under the granular fill, a deposit of fine sand was encountered.

Also, at the location of borehole BH-01-17, the penetration index "N" was measured 4 times during the sampling with the standard split corer in this silt deposit. Values of "N" varying between 4 and 5 were obtained. The density of this deposit can be described as "loose".

Two (2) particle size analysis and one (1) determinations of the natural moisture test were performed on samples taken from this deposit. The results are summarized in Table 5.

Table 5 Results of the sieves analyze on target samples - West Abutment

Boreholes #	Depth (m)	Gravel > 5 mm (%)	Sand < 5mm and > 80µm (%)	Silt and clay < 80µm (%)	Ucsc
F-08-15	3.05 - 3.66	0.0	12.4	87.5	ML
F-08-15	4.88 - 5.49	0.0	52.7	47.3	SM-ML

The results from the analysis and visual examinations of samples indicate that this deposit consists of a materials ranging from a "silt with some sand" to a "sand and silt."

3.3 CLAY DEPOSIT

At the location of the borehole BH-01-17, a clay deposit was encountered immediately below the sand deposit, and below granular fill in BH-02-17. The soil sampling was done to a maximum depth of 8.23 m within this deposit.

DCPT were performed at the location of the borehole BH-01-17 in order to determine the thickness of the deposit. A dense deposit was encountered at a depth of 25 m, followed by a refusal at 28.50 m depth.

This deposit was composed of gray silty clay of medium to high plasticity.

The shear strength of the intact and remodeled clay was measured at the location of the borehole BH-01-17, in accordance with NQ-2501-200 standard, using a controlled deformation Nilcon field vane. The testing was performed down from depths of 4.27 m to 8.27 m. The consistency of the deposit varies generally from very soft to medium.

The measures of the clay consistency limits were performed in the laboratory on two (2) representative samples of this deposit. The results are presented in Table 6 below and in the Appendix 3.

Table 6: Consistency limits results – East abutment

Borehole #	Samples nbr	Depth (m)	W_n (%)	W_l (%)	W_p (%)	I_p (%)	I_l	Uscs
BH-01-17	SS-9	5.33 – 5.94	64.5	49	25	24	1.6	CL
BH-01-17	SS-11	6.86 – 7.47	64.5	40	21	19	2.3	CL
BH-02-17	SS-3	1.22 – 1.83	64.7	85	29	56	0.6	CH

According to the results and the Unified Soil Classification System (USCS), this is an inorganic, medium plastic “CL” to high plasticity clay deposit "CH".

3.4 DENSE DEPOSIT

A dynamic cone penetration tests (DCPT) performed on the borehole BH-01-17 encountered a dense deposit from 25 m depth to a refusal at 28.50 m. The DCPT do not allow the collection of sample to identify the naturel of soils encountered.

4 GROUNDWATER

As mentioned previously, a "Casagrande" piezometers were left in place at borehole BH-01-17, to measure and monitor, if necessary, the level of groundwater in the subsoil. Table 7 shows the result obtained during the measurement of water level.

Table 7: Water level measured in the piezometer

Borehole #	Type	Reading date	Depth (m) [elevation (m)]	
			Water level	Bottom of piezometers
BH-01-17	Casagrande	2017-11-30	Ground Surface [119.93]	3.81 [116.12]

The water level was approximately 1.8 m below bridge deck (El. 119.50 m).

It is important to note that the level of groundwater can be influenced by several factors including, precipitation, melting of snow and changes to the physical environment. It also should be mentioned that the stabilization of the water level in the piezometer installed in the clay deposit requires several weeks.

5 DISCUSSION AND RECOMMANDATIONS

5.1 GENERAL REMARKS

The project consists in the reconstruction of Trail #50 pedestrian bridge, located on Pine Road across Meech Creek, in Wakefield, Quebec. As-built plan indicate the bridge has a approximate 7 m span, and 4.2 m width.

According to the information received, it is planned to use a similar type of foundation than the existing ones. The location of the foundations will be moved up to 8 m to increase bridge span and reduce abutment encroachment of the creek.

The soil layers encountered at BH-01-17 consists of a granular fill down to a depth of about 1.2 m, followed by natural sand to a depth of 4 m and followed by clay deposit. The natural soil becomes very dense at a depth of 25 m.

At the location of the borehole BH-02-17, performed further on the trail, a similar granular fill was intercepted, followed by a clay deposit encountered this time at 1.2 m. This borehole ended at 3 m depth in the clay deposit.

On November 30th, the groundwater level was at surface level at the borehole BH-01-17. It was approximately 0.4 m higher than the water level of the river.

Based on the available information and the results obtained on site, our geotechnical recommendations and comments for this project are presented in the following sections.

The recommendations are prepared according to the *"Norme relative aux ponts sur les terres du domaine de l'État* ", Published by the Ministry of Forests, Wildlife and Parks, updated on April 5, 2016.

5.2 EXCAVATION AND DEWATERING

5.2.1 Overview

The excavations required to reach the foundation level can be done in open trenches. Given that the contractor's working method is unknown at this moment and that they are temporary, their stability as well as the safety of the workers and structures, are the responsibility of the contractor when these are temporary slopes.

Considering a foundation level of 116.00 m to 117.00 m, it will be lower than the recorded level of the water measured at 119.50 m. In the case of excavations done below the level of the river, it will be necessary to set up an adequate cofferdam, prior to the work to be undertaken. The recommendations in section 2.3.3 of the guide on "*Norme relative aux ponts sur les terres du domaine de l'État*", will have to be followed.

As an indication and for the purpose of volumes estimating of excavated soil, the slopes of temporary excavation slopes may be considered as 2.0 horizontally for 1.0 vertical in the fill materials and the in clayed soil.

Excavation slopes must be adjusted on site according to the site conditions observed during the work and also according to the contractor's working methods. The surfaces of the temporary excavation slopes shall be uniform and free of pebbles and loose blocks. Finally, it is understood that in the presence of instability, the slopes must be adjusted.

If any unsupported excavations remain open for a long periods of time, it is recommended that a daily inspections be completed by a specialized geotechnical personnel to identify any risk of instability and determine if any correction is required.

It is important to consider that the use of trench boxes is not an effective land support system. They should be considered only as a protection system for workers.

It is recommended to avoid parking heavy vehicles at the top of any excavations at a distance less than the depth of the excavation. It is also recommended to avoid circulating with a vehicle at the top of excavations within a distance of less than the depth of the excavations to minimize the vibrations. These conditions must be respected at all times unless special investigations are carried for each specific case. It is the same when structures are located near the excavations.

In all cases, the Quebec Construction Code and the requirements of the CNESST must be respected at all times during the excavations.

5.2.2 Temporary Dewatering of Excavations

Given the groundwater observed on November 30th, 2017 (see section 4), excavations required for the construction of the foundations would be carried out below the groundwater level. In addition, seepage and runoff can also accumulate in the bottom of excavations.

It is essential to provide an adequate and effective temporary drainage system throughout the construction of the foundations to eliminate any runoff and seepage as they accumulate in the bottom of the excavations in order to carrying out the work in dry conditions.

It is also recommended to maintain the groundwater level at least 300 mm below the level of the bottom of the excavation during the construction. This procedure will ensure the stability of the temporary excavation slopes and prevent the phenomenon of buoyancy, which would have the effect of destabilizing the bottom of the excavation.

Therefore, it is recommended that an appropriated pumping system using wells or trenches be provided in order to lower groundwater level and evacuate the runoff and seepage water that can accumulate at the bottom of the excavations, depending on the weather conditions during the construction, in order to carry out the work in a dry and safe environment.

5.2.3 Reuse of Excavated Materials

The Granular materials from the road fill can be reused for backfilling excavations to the level of the subgrade, as long as their water content is adequate during the compaction. Backfill material must be free of blocks and pebbles (diameter greater than 200 mm) that may be found in the existing abutment and trail fill.

It is recommended that excavated granular material be analysed in order to validate the potential of reuse.

If it becomes impossible to reuse the excavation material, the backfilling can then be completed with granular of MG-112 (CCDG) materials down to the subgrade level. These materials must be compacted to at least 95% of the modified Proctor (NQ 2501-255).

Reuse of backfill soil also remains subject to environmental regulations in effect with the MDDELCC.

5.3 FOUNDATIONS

5.3.1 Frost Depth

According to the Environment Canada database, the average freezing index is 1,008 ° C-day in the Chelsea area , the nearest meteorological station for which this index is available. The corresponding anticipated depth for frost penetration in the soils is therefore estimated at 1.8 m. Therefore, the bridge foundations exposed to freezing (mainly on the river side) must be covered with soil to a minimum thickness of 1.8 m in order to protect them from the frost action.

5.3.2 Surface Preparation

Soils containing deleterious and / or unsuitable materials, must be excavated at the location of the projected constructions. In addition, it is recommended that the bottom of excavation for the foundation be checked and approved by a geotechnical engineer or his representative to detect any soft or unsuitable surface or construction and to recommend the appropriate corrections if required.

Given the heterogeneous nature of the backfill, it is recommended that any existing backfill be excavated until the sand and silt deposit (above elevation 115.89 m) or undisturbed clay deposit (below elevation 115.89 m) is reached.

Special precaution must be taken to avoid any remoulding of the clay deposit encountered at the bottom of the excavations. For this purpose, it is recommended to put in place a cushion of granular material with a thickness of at least 300 mm. This cushion should consist of a compactable granular fill, free of organic matter or blocks, to be installed by layer of 150 mm and densified to 92% of the reference density (modified Proctor). The compaction must be done by minimizing the vibrations transmitted to the underlying deposit, through the use of light compaction equipment, such as a vibrating plate.

An alternative to the granular material would be the use of a layer of lean concrete or unshrinkable fill placed directly under the foundations installed on the clay deposit, and, immediately at the end of the excavations.

Excavations in the clay deposit should be carried out using a wide tooth bucket.

In the case of unstable soils, these shall be excavated and replaced with materials of similar type and compacted.

The surface of the excavation must be free of any particle of diameter greater than 100 mm. The bottom should also be free of ice or frozen soils. Protective measures against soil freezing could also be considered.

5.3.3 Geotechnical Resistance at Ultimate Limit States (ULS)

The calculation of the shallow foundations must done according to the ultimate limit state method. The ultimate geotechnical resistance (q_{ult}) can be evaluated from the following relationship in accordance with "*Canadian Manual engineering foundation – 4th Edition, 2013*":

$$ULS = c\ N_c\ S_c\ I_c\ \beta_c + q'\ N_q\ S_q\ I_q\ \beta_q + 0{,}5\ \gamma\ B\ N_\gamma\ S_\gamma\ I_\gamma\ \beta_\gamma$$

ULS = Ultimate limit state

q' = Soil pressure at the foundation level

c' = effective cohesion = 5 for the long-term analysis and c = Cu for the short-term analysis

In the case of an eccentric load, the width of the footing must be modified to take into account the eccentricity and make a concentric load on the footing and the actual width B' and a length L' would be :

$B' = B - 2e_B$, but less than L'

$L' = L - 2e_L$

e : The eccentricity of the load

S_c, S_q, S_γ are shape coefficients for taking into account of the geometry of the footing:

$S_c = S_q = 1 + (B'/L')\ (N_q/N_c)$

$S_\gamma = 1 - 0{,}4\ (B'/L')$

I_c, Iq, I_γ are slope coefficients to take into account the inclination of the load:

$I_c = I_q = (1 - \gamma/90°)^2$

$I_\gamma = (1 - \gamma/\gamma')^2$

γ: is the resulting angle force relative to the vertical

β_c, β_q and β_γ are shape coefficients to into take account the ground slope around the foundation (β, expressed in radians, valid for $\beta < \pi / 4$) :

$\beta_c = \beta_q - (1 - \beta_q) / (Nc \tan \gamma)$

$\beta_q = (1 - \tan \beta)^2$

$\beta_\gamma = (1 - \tan \beta)^2$

The geotechnical parameters presented in Table 8 are used for the calculating of the geotechnical resistance to the ultimate limit state for shallow foundations sitting on the clay deposit.

Table 8: Geotechnical Parameters – Clay Deposit

Parameters	Clay deposit (short term)	Clay deposit (long term)
Undrained shear strength (c_u)	38 kPa	5 kPa
Effective internal friction angle (ϕ')	0°	28°
Bearing capacity factor (Nc)	5,14	26
Bearing capacity factor (Nq)	1	15
Bearing capacity factor ($N\gamma$)	0	11
Volumetric weight (γ)	17 kN/m³	17 kN/m³
Effective volumetric weight (γ')	7 kN/m³	7 kN/m³

To evaluate the factored resistance at the ultimate, the Canadian Code on Highway Bridge (CSA S6-14) recommends the use of a coefficient of 0.50.

5.3.4 Geotechnical Resistance at Serviceability Limit States (SLS)

Based on the information collected during the investigations, it is expected that for a wooden crib caisson foundations, of a dimension similar to the existing foundations (approx. 3.5 m x 4.2 m), installed at a depth of 3.0 to 4.0 m. The foundation will be founded either on the sand deposit of less than 1 m of thickness, or the clay deposit.

A geotechnical resistance at Serviceability Limit States (SLS) of **150 kPa** can be considered for a maximum settlement of **50 mm**. For a maximum settlement of **25 mm**, a geotechnical resistance at Serviceability Limit States (SLS) of **75 kPa** can be considered.

It will be essential that the preparation under the foundation is done according to the recommendations of Section 5.3.2 of this report.

Also, no grade rising of the site has been considered in the calculation presented above. If this becomes necessary, resistance geotechnical resistance at Serviceability Limit States would have to be revised.

5.3.5 Factored Horizontal Reaction of the Caissons

Table 9 presents the friction angles for the different interfaces.

Table 9 : Angle of Friction (δ) for the Different Interfaces

Interface	Friction angle (δ)
Cushion (MG 20) – Soil (clay)	14°
Backfill (MG 112) – Cushion (MG 20)	18°
Caisson – Backfill (MG 112)	18°
Caisson – Cushion (MG 20)	18°

5.3.6 Earth Pressure

The cribwork caissons will be subject to earth pressure caused by the backfill behind them. In general, since the backfill material in place behind the wall must be compacted to a distance less than 3 meters of structures, the stress distribution shown in Figure 2 should be considered.

Figure 2 : Stress distribution of backfills

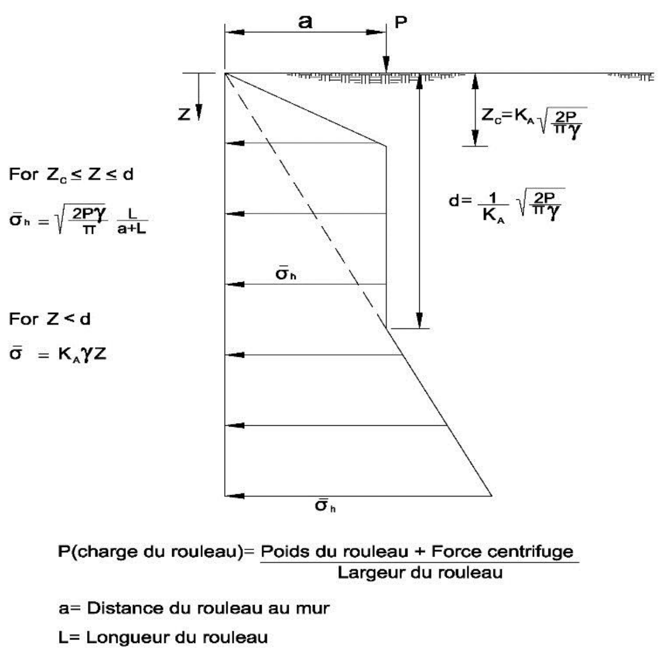

Table 10: Geotechnical Parameters for the Earth pressure on a Engineered Fill

Parameters	Mg 20 or mg 112 type backfill
Volumetric weight (γ) kN/m^3	20
Internal angle of friction (ϕ')	34
Active earth pressure coefficient K_a*	0.28
Passive earth coefficient coefficient K_p*	3.54
Coefficient of earth pressure at-rest K_o*	0.44
* For vertical wall and a horizontal supported soil surface	

5.4 STABILITY OF THE ABUTMENTS

The abutments stability and overall stability of the walls will be verified once the concepts will be defined, and under a separate mandate.

5.4.1 Abutments Protection Against Scouring

Since the footing might be placed above the riverbed of the Meech Creek, an adequate protection against scour will be necessary. This protection could be a rock protection of a sufficient size (300-500 mm in diameter or larger) in function of the water flow under the bridge, and other parameters.

Also, additional protection must be provided to prevent the granular material constituting the cushion to build between the bottom of the foundation and the riverbed materials doesn't scour.

5.5 SEISMIC PARAMETERS

5.5.1 Spectral Acceleration

The spectral acceleration values for different periods as well as the value of the peak ground acceleration (PGA) for different cities and municipalities are given in the National Building Code of Canada (NBC). In the Wakefield area, for a probability of being over of 2% over 50 years, the spectral acceleration data and peak ground acceleration for a Class C site are indicated in Table 11 for the 2010 and the 2015 editions of the NBC.

Table 11: Spectral and Peak Ground Acceleration - Site Class C

Site locataion	Nbc version	Seismic data				
		Sa(0,2)	Sa(0,5)	Sa(1,0)	Sa(2,0)	Pga (g)
Wakefield, Québec 45.5709°N 75.8871°W	2010	0.630	0.308	0.137	0.046	0.319
	2015	0.413	0.224	0.112	0.054	0.256
* The seismic hazard values of the National Building Code can be determined at the following link: http://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/index-en.php						

The designer must adjust the spectral accelerations according to the location site listed in the following section. The designer must refer to section 4.1.8 of the 2015 NBC.

5.5.2 Location Category

Based on the information obtained during this study, the location category to be considered would be a type "**E**", according to the descriptions provided in Section 4.4.3.2 of the "Canadian Design Code highway bridge, S6-14." The design must refer to this standard for the site coefficient values F (t), F (AMH) and F (VMS).

5.5.3 Potential Assessment Soil Liquefaction

The method used to assess the liquefaction potential is that of the *Greater Vancouver Liquefaction Task Force Report* (2007). Soils with a plasticity index above 7% are considered unsusceptible to liquefaction. The clay deposit, having a plasticity indices higher than 7%, this deposits will have no potential of liquefaction under a seismic event, based on the NBC 2010 and 2015.

Appendix 1 Scope of the geotechnical study

SCOPE OF THE GEOTECHNICAL STUDY

1.0 Characteristics of soil and rock

The soil and rock characteristics described in this report originate from geotechnical investigations conducted within a given period and correspond to the nature of the terrain only at the specific locations where these investigations were carried out.

Soil and rock formations have natural variations. The limits between the different formations presented in the sounding logs must therefore be considered as transitions between the formations rather than set boundaries. The precision of these limits depends on the type and number of soundings, the sounding methods used, as well as sampling frequency and methods.

The descriptions of the samples taken are based on recognized identification and classification methods used in geotechnics. They can call into play the judgement and interpretation of the personnel who carried out the examination of materials and can be presumed to be accurate and correct in keeping with current best practices in the field of geotechnics. Finally, if tests were carried out, the results of these tests apply solely to the samples tested, as described in this report.

The properties of the soil and rock can undergo significant modifications in the wake of construction activities such as excavation, blasting, pile driving or drainage activities, carried out on the site under study or an adjacent site. They can also be indirectly modified by the exposure of the soil or rock to freezing or weather stresses.

2.0 Groundwater

The groundwater conditions presented in this report apply only to the site under study. The accuracy and representation of these conditions must be interpreted based on the type of instrumentation used, as well as the period, duration, and number of observations carried out. These conditions can vary depending on precipitation, the seasons and, ultimately, the tides. They can also vary as a result of construction activities or the modification of physical elements on the site under study or in its vicinity. The problematic of ferrous ochre and its effects is not covered in this report.

3.0 Use of the report

The comments and recommendations contained in this report are intended primarily for the project's design team. The number of soundings required to identify all of the underground conditions that could impact construction costs, techniques, the choice of equipment and planning of operations could be greater than the number required for design purposes. All contractors bidding on or carrying out the work on the site under study must undertake their own interpretation of the results of the soundings and, if need be, carry out their own investigations to determine how site conditions could influence their operations or work methods.

Any modifications to the design, position and elevation of the works must be quickly communicated to Englobe, allowing the validity of the recommendations presented to be verified. Complementary site or laboratory work could ultimately be required.

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4.0 Project tracking

The interpretation of the on-site and laboratory results obtained, as well as the recommendations presented in this report, apply solely to the site under study and to the information available about the project at the time this report was drafted.

Information available concerning the site and groundwater conditions increases as construction work progresses. As site conditions were interpreted and correlated between sounding points, Englobe should be allowed to verify these conditions, during site visits conducted as work progresses, in order to confirm the information provided by the drillings soundings. If it is not possible for us to conduct these verifications, Englobe shall assume no responsibility for geotechnical interpretations by third parties concerning recommendations contained in this report, particularly if the design has been modified or if site conditions different from those described in this report are encountered. The identification of such changes requires experience and must be carried out by an experienced geotechnical engineer.

5.0 Environment

The information contained in this report does not cover the environmental aspects of the site conditions, as these aspects were not included in the study mandate.

Appendix 2 Explanative notes, borehole logs

EXPLANATION NOTE ON SOUNDING LOGS

The following sounding logs summarize soils and rock geotechnical properties as well as ground water conditions, as collected during field work and/or obtained from laboratory tests. This note explains the different symbols and abbreviations used in these logs.

STRATIGRAPHIC UNITS

Elevation/Depth: Reference to the geodesic elevation of the soil or to a bench mark of arbitrary elevation, at the location of the sounding. Depth of the different geological boundaries as measured from ground surface. On the left, the scale is in meters while on the right, it is in feet.

Description of the stratigraphic units: Every geological formation is detailed.

The proportion of the different elements of the soil, defined according to the size of the particles, is given following the classification hereafter. The relative compactness of cohesionless soils is defined by the "N" index of the Standard Penetration Test. The consistency of cohesive soils is defined by their shear resistance.

Classification	Particle size (mm)
Clay	< 0.002
Clay and silt (undifferentiated)	< 0.08
Sand	0.08 to 5
Gravel	5 to 80
Cobble	80 to 300
Boulder	> 300

Descriptive terminology	Proportion (%)
"Traces" (tr.)	1 to 10
"Some" (s.)	10 to 20
Adjective (ex.: sandy, silty)	20 to 35
"And" (ex.: sand and gravel)	35 to 50

Compactness of cohesionless soils	Standard Penetration Test index ("N" value), ASTM D-1586 (blows for a 300mm penetration)
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	> 50

Consistency of cohesive soils	Undrained shear strength (kPa)
Very soft	< 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very stiff	100 to 200
Hard	> 200

Plasticity of cohesive soils	Liquid limit (%)
Low	< 30
Medium	30 to 50
High	> 50

Sensitivity of cohesive soils	$S_t = (C_u/C_{ur})$
Low	$S_t < 2$
Medium	$2 < S_t < 4$
High	$4 < S_t < 8$
Extra-sensitive	$8 < S_t < 16$
Quick (sensitive) clay	$S_t > 16$

Classification of rock	RQD (%)
Very poor quality	< 25
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

SYMBOLS

TOP SOIL	SAND	COBBLE
BACKFILL	SILT	BOULDER
GRAVEL	CLAY	ROCK

WATER LEVEL

This column shows the ground water level, as measured at a given time during the geotechnical investigation. The details of the installation (type and depth) are also illustrated in this column.

SAMPLES

Type and number: Each sample is labelled in accordance with the number of this column and the given notation refers to samples types.

Sub-sample: When a sample contains two or more different stratigraphic units, it is sometimes necessary to separate it and create sub-samples. This column allows for the identification of the latter and the association to *in situ* or laboratory measurements to these sub-samples.

Condition: The position, length and condition of each sample are shown in this column. The symbol shows the condition of the sample, following the legend given on the sounding log.

Size: This column indicates the split spoon sampler size.

"N" index The standard penetration index shown in this column is expressed with the letter "N". This index is obtained with the Standard Penetration Test. It corresponds to the number of blows required to drive the last 300mm of the split spoon, using a 622 Newton hammer falling freely from a height of 762mm (ASTM D-1586). For a 610mm long split spoon, the "N" index is obtained by adding the number of blows required for the driving of the 2nd and 3rd 150mm of the split spoon. Refusal (R) indicates a number of blows greater than 100. A set of numbers such as 28-30-50/60mm indicates that the number of blows required to drive the 1st and 2nd 150mm of the split spoon are respectively 28 and 30. Moreover, it indicates that 50 blows were necessary to get a penetration of 60mm, whereupon the test was suspended.

RQD index: Rock Quality Designation index: This index is defined as the ratio between the total length of all rock cores of 100mm and more in length over the total length of the core run. The RQD index is an indirect measurement of the number of "natural" fractures and of the amount of the alteration in a rock mass.

TESTS

Results: This column shows, for the corresponding depth, the results of tests carried out in the field or in the laboratory (shear strength, dynamic penetration, Atterberg limits with the cone, etc.). For more information, please refer to the legend in the upper part of the sounding log. However, an abbreviation indicating the type of analysis performed is shown next to the sample tested.

Graph: This graph shows the undrained shear strength resistance of cohesive soils, as measured *in situ* or in the laboratory (NQ 2501-200). It is also used to present the Dynamic Cone Penetration Test (NQ 2501-145) results.

Moreover, this graph is used for the representation of the water content and Atterberg limits test results.

Englobe

Client : **National Capital Commission**

BOREHOLE REPORT

File n°: **B-0018588-1**
Borehole n°: **BH-01-17**
Date: From **2017-11-27** to **2017-11-28**

Project: **Soil Investigation - Trail #50 Bridge Replacement**

Location: **Trail #50 Bridge in Gatineau Park Near Parking P50, Pine Road, Wakefield, Quebec**

Coordinates (m): **MTM, NAD3 ZONE 9**
North 5048089,0 (Y)
East 352638,0 (X)
Elevation **119,93 (Z)**
Bedrock: m End depth: 28,50 m

Sample condition

Intact | Remoulded | Lost | Core

Organoleptic soil examination:

Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	Englobe Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits	O.M.	Organic Matter (%)	▼	Water Level
W_L	Liquid Limit (%)	K	Permeability (cm/s)	N	Std Penetration test (blows/300mm)
W_P	Plastic Limit (%)	UW	Unit Weight (kN/m³)	N_C	Dyn. Penetration test (blows/300mm) ●
I_P	Plasticity Index (%)	A	Absorption (l/min. m)	σ'_P	Preconsolidation Pressure (kPa)
I_L	Liquidity Index	U	Uniaxial Compressive strength (MPa)	SCI	Soil Corrosivity Index
W	Natural Water Content (%)	RQD	Rock Quality Designation (%)	EDM	Deformation Modulus (GPa)
GS	Grain Size Analysis	CA	Chemical Analysis	σ'_T	Brazilian test (MPa)
S	Hydrometer analysis	P_L	Limit Pressure (kPa)		Undrained shear strength — Field / Laboratory
R	Refusal	E_M	Pressuremeter Modulus (MPa)	C_U	Undisturbed (kPa) ▲ ■
VBS	Methylene Blue Value	E_r	Modulus of subgrade reaction (MPa)	C_{UR}	Remoulded (kPa) △ □
WR	Weight of Rods	SP_O	Segregation Potential (mm²/H °C)		

R.F.

STRATIGRAPHY

ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION
119,93 / 0,00	***Granular Fill:*** Sand and gravel, traces of silt.
119,32 / 0,61	Sand, some gravel and silt, grey, moist.
118,71 / 1,22	***Sand Deposit:*** Silty sand, traces of gravel, grey, moist.
118,10 / 1,83	Grey fine sand and silt, loose consistency, saturated.
116,88 / 3,05	Grey fine sand and some silt, loose consistency, saturated.
115,89 / 4,04	***Clay Deposit:*** Grey silty clay, firm consistency, medium plasticity.
115,66 / 4,27	Beginning of the scissometer tests on site at 4.27 m.
114,66 / 5,27	Soft to very soft consistency
112,66 / 7,27	Medium consistency
111,70 / 8,23	End of sampling at a depth of 8.23 m and beginning of the dynamic cone penetration test.

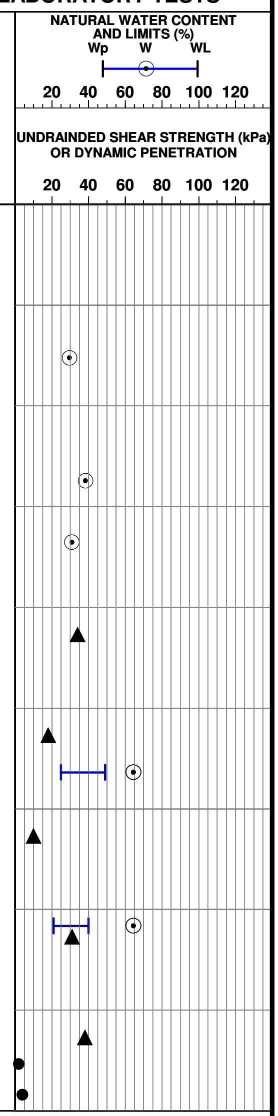
WATER LEVEL (m) / DATE: el. 119,93 m 2017-11-30

SAMPLES

TYPE AND NUMBER	SUB-SAMPLE	RECOVERY %	Blows/150mm	"N" or RQD	Odor	Visual
SS-1		12	3-6 5-4	11	I	I
SS-2		8	2-3 2-2	5	I	I
SS-3		67	2-2 2-2	4	I	I
SS-4		54	2-2 3-2	5	I	I
SS-5		33	2-3 1-1	4	I	I
SS-6		87	3-2 2-6	4	I	I
SS-7	A	100	1-0 1-1	1	I	I
	B				I	I
SS-8		100	1-2 2-1	4	I	I
SS-9		100	0	PDT	I	I
TM-10					I	I
SS-11		92	0	PDT	I	I
SS-12		100	0	PDT	I	I

FIELD AND LABORATORY TESTS

RESULTS



Vertical Scale = 1 : 75

Remarks: Ground protector filled with water at the moment of the water reading.

Borehole type:
Boring equipment: **TM-2**

Prepared by: **S. Séguin, tech.** | Approved by: **T. Lampron, Eng.** | 2018-01-17 | Page: 1 of 3

Englobe

Client : **National Capital Commission**

BOREHOLE REPORT

File n°: **B-0018588-1**
Borehole n°: **BH-01-17**
Date: From **2017-11-27** to **2017-11-28**

Project: **Soil Investigation - Trail #50 Bridge Replacement**

Location: **Trail #50 Bridge in Gatineau Park Near Parking P50, Pine Road, Wakefield, Quebec**

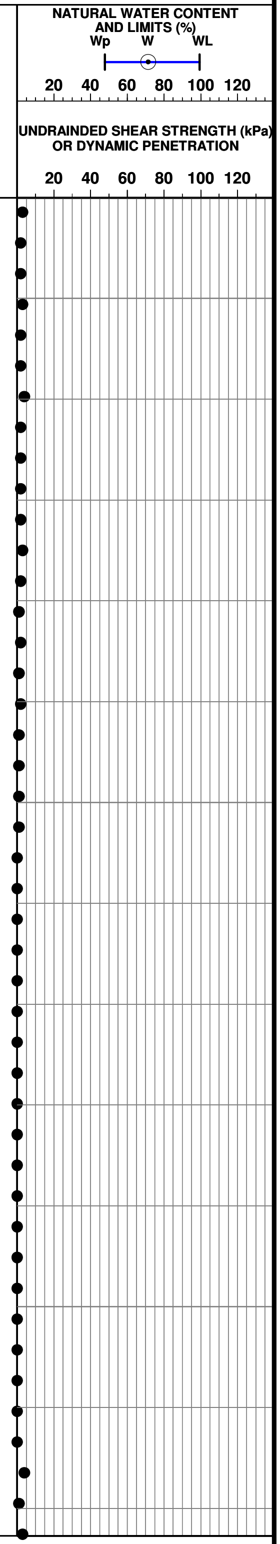
Coordinates (m): **MTM, NAD3 ZONE 9**
North 5048089,0 (Y)
East 352638,0 (X)
Elevation **119,93 (Z)**
Bedrock: m End depth: 28,50 m

R.F.

Vertical Scale = 1 : 75

DEPTH - ft	DEPTH - m	ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION	SYMBOLS	WATER LEVEL (m) / DATE	TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Organo. Exam: Odor	Organo. Exam: Visual	RESULTS
30			End of the scissometer tests on site at 8.27 m.												
31															
32															
33	10														
34															
35															
36	11														
37															
38															
39	12														
40															
41															
42															
43	13														
44															
45															
46	14														
47															
48															
49	15														
50															
51															
52	16														
53															
54															
55															
56	17														
57															
58															
59	18														
60															
61															
62	19														
63															
64															
65															
66	20														
67															
68															
69	21														
70															
71															
72	22														

FIELD AND LABORATORY TESTS



Remarks: Ground protector filled with water at the moment of the water reading.

Borehole type:

Boring equipment: **TM-2**

Prepared by: **S. Séguin, tech.**

Approved by: **T. Lampron, Eng.**

2018-01-17

Englobe

Client : **National Capital Commission**

BOREHOLE REPORT

File n°: **B-0018588-1**
Borehole n°: **BH-01-17**
Date: From **2017-11-27** to **2017-11-28**

Project: **Soil Investigation - Trail #50 Bridge Replacement**

Location: **Trail #50 Bridge in Gatineau Park Near Parking P50, Pine Road, Wakefield, Quebec**

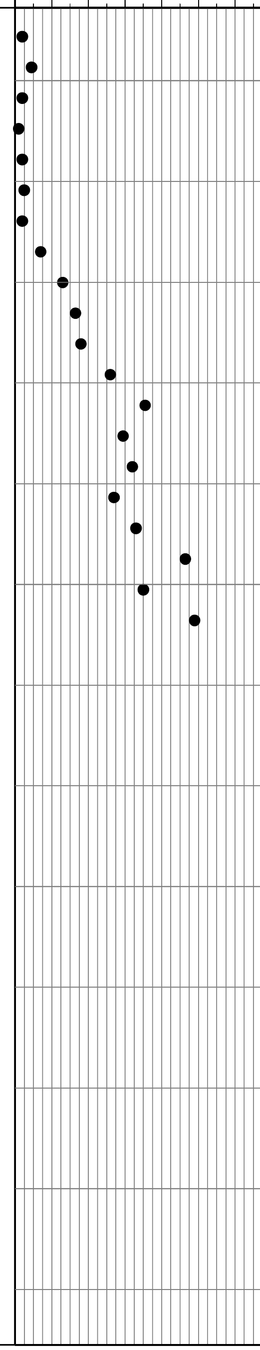
Coordinates (m): **MTM, NAD3 ZONE 9**
North 5048089,0 (Y)
East 352638,0 (X)
Elevation **119,93 (Z)**
Bedrock: m
End depth: 28,50 m

DEPTH - ft	DEPTH - m	ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION (STRATIGRAPHY)	SYMBOLS	WATER LEVEL (m) / DATE	TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Organo. Exam: Odor	Organo. Exam: Visual	RESULTS
74–93	23–28	91,43													
94		28,50	End of the dynamic penetration test at 28.5 m after obtaining a refusal on dense soils, block or probable bedrock.												
95–116	29–35														

FIELD AND LABORATORY TESTS

NATURAL WATER CONTENT AND LIMITS (%) — Wp W WL — 20 40 60 80 100 120

UNDRAINED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION — 20 40 60 80 100 120



R.F.

Vertical Scale = 1 : 75

Remarks: Ground protector filled with water at the moment of the water reading.

Borehole type:
Boring equipment: **TM-2**

Prepared by: **S. Séguin, tech.**
Approved by: **T. Lampron, Eng.**
2018-01-17

Englobe

Client : **National Capital Commission**

BOREHOLE REPORT

File n°: **B-0018588-1**
Borehole n°: **BH-02-17**
Date: From **2017-11-27** to **2017-11-27**

Project: **Soil Investigation - Trail #50 Bridge Replacement**

Location: **Trail #50 Bridge in Gatineau Park Near Parking P50, Pine Road, Wakefield, Quebec**

Coordinates (m): **MTM, NAD3 ZONE 9**
North 5048121.0 (Y)
East 352666.0 (X)
Elevation **121.70 (Z)**
Bedrock: m End depth: 3.05 m

Sample condition: Intact; Remoulded; Lost; Core

Organoleptic soil examination:
Visual aspect: Non-existent(N); Disseminated(D); Soaked(S)
Odor: Non-existent(N); Light(L); Medium(M); Persistent(P)

Sample type

SS	Split Spoon
TM	Thin wall Tube
PS	Piston Tube
RC	Rock core
AS	Auger
MA	Bulk sample
TU	Transparent tube
PW	Englobe Mega-Sampler
FG	Frozen ground

Tests

L	Consistency Limits	**O.M.**	Organic Matter (%)
W_L	Liquid Limit (%)	**K**	Permeability (cm/s)
W_P	Plastic Limit (%)	**UW**	Unit Weight (kN/m³)
I_P	Plasticity Index (%)	**A**	Absorption (l/min. m)
I_L	Liquidity Index	**U**	Uniaxial Compressive strength (MPa)
W	Natural Water Content (%)	**RQD**	Rock Quality Designation (%)
GS	Grain Size Analysis	**CA**	Chemical Analysis
S	Hydrometer analysis	**P_L**	Limit Pressure (kPa)
R	Refusal	**E_M**	Pressuremeter Modulus (MPa)
VBS	Methylene Blue Value	**E_r**	Modulus of subgrade reaction (MPa)
WR	Weight of Rods	**SP_O**	Segregation Potential (mm²/H °C)

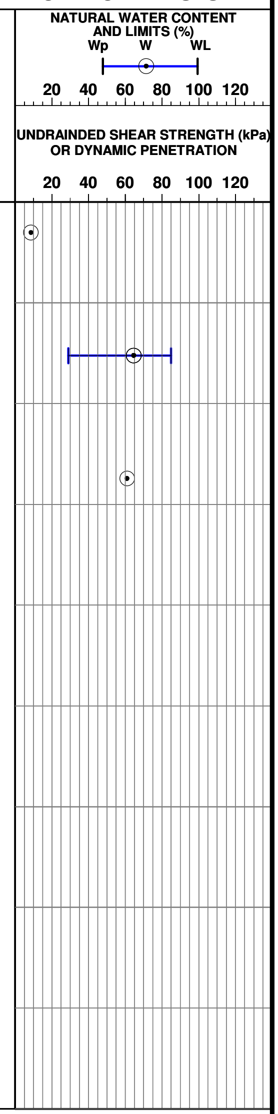
▼	Water Level
N	Std Penetration test (blows/300mm)
N_C	Dyn. Penetration test (blows/300mm) ●
σ'_P	Preconsolidation Pressure (kPa)
SCI	Soil Corrosivity Index
EDM	Deformation Modulus (GPa)
σ'_T	Brazilian test (MPa)

Undrained shear strength (Field / Laboratory)

		Field	Laboratory
C_U	Undisturbed (kPa)	▲	■
C_{UR}	Remoulded (kPa)	△	□

R.F.

DEPTH - ft	DEPTH - m	ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION	SYMBOLS	WATER LEVEL (m) / DATE	TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Odor	Visual	RESULTS
		121.70 0.00	***Granular Fill:*** Well-graded, gravelly sand with some silt, dry.			SS-1		Remoulded		54	13-9 9-8	18	I	I	GS W = 8.7
		121.09 0.61	Sand, some gravel, traces of silt, grey.			SS-2		Remoulded		17	5-4 3-2	7	I	I	
		120.48 1.22	***Clay Deposit:*** Grey silty clay with traces of sand, soft apparent consistency, very humid.			SS-3		Remoulded		100	1-2 3-4	5	I	I	L W = 64.7 W_L = 85 W_P = 29
		119.87 1.83	Silty clay, grey, soft apparent consistency, moist.			SS-4		Remoulded		100	1-1 2-2	3	I	I	
						SS-5		Remoulded		100	2-1 1-2	2	I	I	W = 61.1
		118.65 3.05	End of borehole at a depth of 3.05 m.												



Vertical Scale = 1 : 75

Remarks:

Borehole type: **Hollow Stem Auger**
Boring equipment: **TM-2**

Prepared by: **S. Séguin, tech.**
Approved by: **T. Lampron, Eng.**
2017-12-21

Appendix 3 Laboratory testing results

Englobe
900, boul. de la Carrière, bur. 100
Gatineau, J8Y 6T5
Téléphone: (819) 778-3143

Testing on Soils, Aggregates and Other Materials

Client : Commission de la Capitale Nationale
Project : NCC Trail #50 Bridge Replacement; Geotechnical Investigation
Location : Gatineau Park, Wakefield, Quebec

Project # : B-0018588-1
Client ref. :
Report # : 1 **Rev.** 0
Page 1 **of** 1

Sampling

Sampling # : 1
Your sampling # :
Material :
Source; location : From borehole

Sampling location : BH-01-17, SS-3; 1.22 - 1.83 m

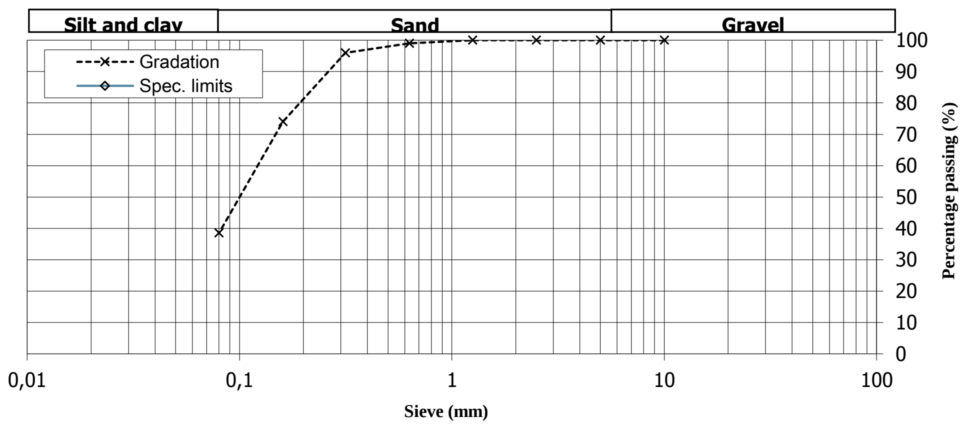
Specification # 1

Reference :
Use :
Calibre :
Class :

Sampling date : 2017-11-27
By : Sylvain Séguin, tech.
Date received : 2017-11-29

Sieve analysis (NQ 2501-025)

SIEVE (mm)	% PASSING	
	REQUIREMENTS	RESULT
112		
80		
56		
40		
31,5		
20		
14		
10		100
5		100
2,5		100
1,25		100
0,630		99
0,315		96
0,160		74
0,080		38,5



Cu : Cc : FM : 0,31 D_{10} : D_{30} : D_{60} : 0,122

Maximum dry density kg/m³	Optimum moisture %	Retained 5 mm %

Proportions from sieve analysis (%)

Cobble : 0,0 Sand : 61,3
Gravel : 0,2 Silt and clay : 38,5

Other testing	Required	Result
Water content (NQ 2501-170) (%)		29,7

Remarks

RESULTS WITH AN ASTERISK DO NOT MEET REQUIREMENTS.

Prepared by : Rock Desjardins, tech. **Date :** 2017-11-30

Approved by : **Tommy Lampron** **Date :**

Englobe
900, boul. de la Carrière, bur. 100
Gatineau, J8Y 6T5
Téléphone: (819) 778-3143

Testing on Soils, Aggregates and Other Materials

Client :	**Commission de la Capitale Nationale**	**Project # :**	**B-0018588-1**
Project :	**NCC Trail #50 Bridge Replacement; Geotechnical Investigation**	**Client ref. :**	
Location :	**Gatineau Park, Wakefield, Quebec**	**Report # :**	**6** **Rev. 0**
			Page 1 of 1

Sampling

Sampling # : 6
Your sampling # :
Material :
Source; location : From borehole

Sampling location : BH-02-17, SS-1; 0.00 - 0.61 m

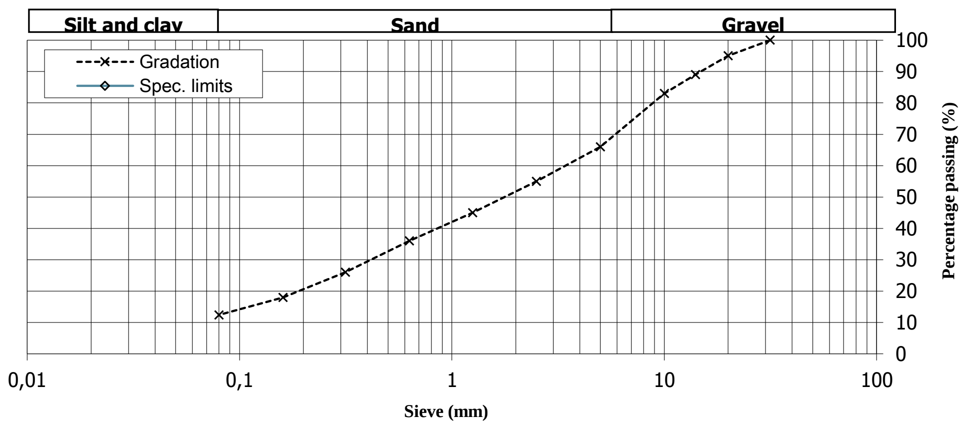
Specification # 1

Reference :
Use :
Calibre :
Class :

Sampling date : 2017-11-27
By : Sylvain Séguin, tech.
Date received : 2017-11-29

Sieve analysis (LC 21-040)

SIEVE (mm)	% PASSING	
	REQUIREMENTS	RESULT
112		
80		
56		
40		
31,5		100
20		95
14		89
10		83
5		66
2,5		55
1,25		45
0,630		36
0,315		26
0,160		18
0,080		12,4



Cu : Cc : FM : 3,54 D_{10} : D_{30} : 0,413 D_{60} : 3,491

Maximum dry density	Optimum moisture	Retained 5 mm
kg/m³	%	%

Proportions from sieve analysis (%)

Cobble : 0,0 Sand : 53,3
Gravel : 34,3 Silt and clay : 12,4

Other testing	Required	Result
Water content (LC 21-201) (%)		8,7

Remarks

RESULTS WITH AN ASTERISK DO NOT MEET REQUIREMENTS.

Prepared by :	**Date :**	**Approved by :**	**Date :**
Rock Desjardins, tech.	2017-12-04	**Tommy Lampron**	

Englobe

900, boul. de la Carrière, bur. 100
Gatineau, J8Y 6T5
Téléphone: (819) 778-3143

Testing on Soils, Aggregates and Other Materials

Client :	**Commission de la Capitale Nationale**	**Project #** :	**B-0018588-1**
Project :	**NCC Trail #50 Bridge Replacement; Geotechnical Investigation**	**Client ref.**	
Location :	**Gatineau Park, Wakefield, Quebec**	**Report #** :	**3** **Rev. 0**
			Page 1 of 1

SAMPLING

Source	: From borehole		
Sampling #	: 3 Your sample # :	Sampled by	: Sylvain Séguin, tech.
Borehole #	: BH-01-17, SS-6	Sampling date	: 2017-11-27
Depth	: 3.05 - 3.66 m	Date received	: 2017-11-29
Location	:	Relative density of particles < 2 mm :	2.700(estimated)

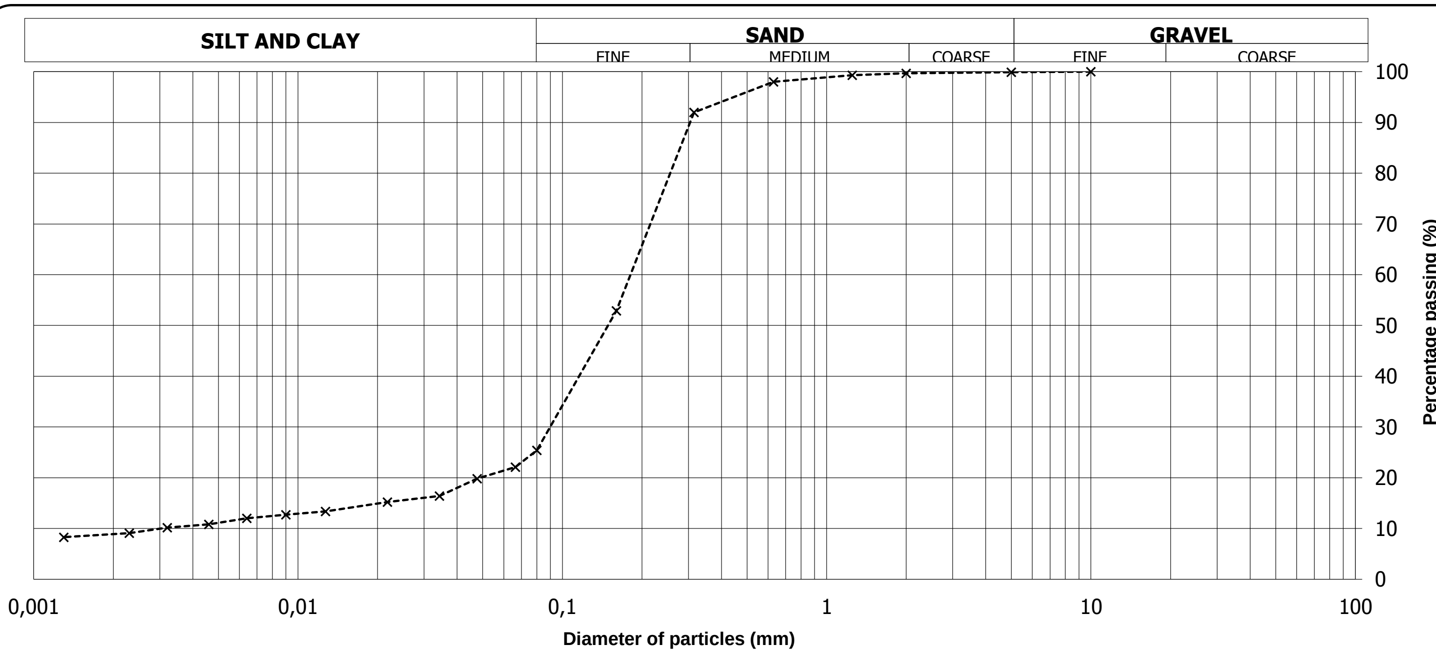
Sieve analysis (NQ 2501-025)		Sedimentation analysis (NQ 2501-025)	
Sieve	% passing	Equivalent diameter	% passing
112 mm			
80 mm		66,5 µm	22,1
56 mm		47,6 µm	19,8
40 mm		34,3 µm	16,4
31,5 mm		21,8 µm	15,2
20 mm		12,7 µm	13,4
14 mm		9,0 µm	12,7
10 mm	100	6,4 µm	12,0
5 mm	100	4,6 µm	10,8
2 mm	100	3,2 µm	10,2
1,25 mm	99	2,3 µm	9,1
0,630 mm	98	1,3 µm	8,3
0,315 mm	92		
0,160 mm	53		
0,080 mm	25,4		

OTHER TESTING	RESULT
Water content (NQ 2501-170) (%)	31,0

REMARKS

Proportions from sieve analysis

Cobble :	0,0	Sand:	74,5
Gravel :	0,1	Silt :	16,5
		Clay :	8,9



D_{10} : 0,003 D_{30} : 0,090 D_{60} : 0,181

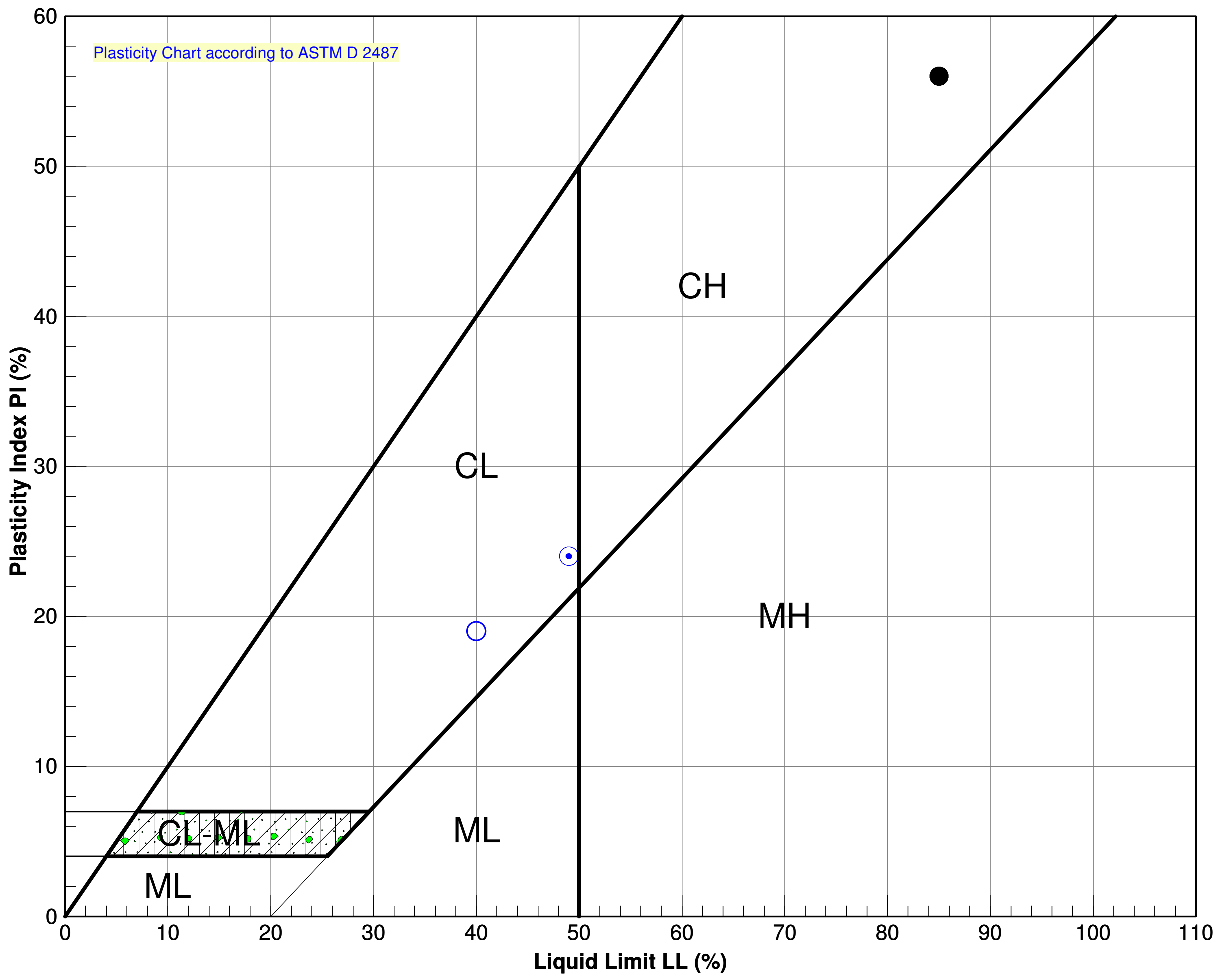
Prepared by :	**Date :**	**Approved by :**	**Date :**
Rock Desjardins, tech.	2017-12-05	**Tommy Lampron**	

R.F. Style_LVM\Lab\Lab_Geotec_80 Lab_Limite_LVM_AN.STY - Printed: 2017-12-14 14:37:34

PLASTICITY CHART

Project : **Soil Investigation - Trail #50 Bridge Replacement** Figure n° : **1**

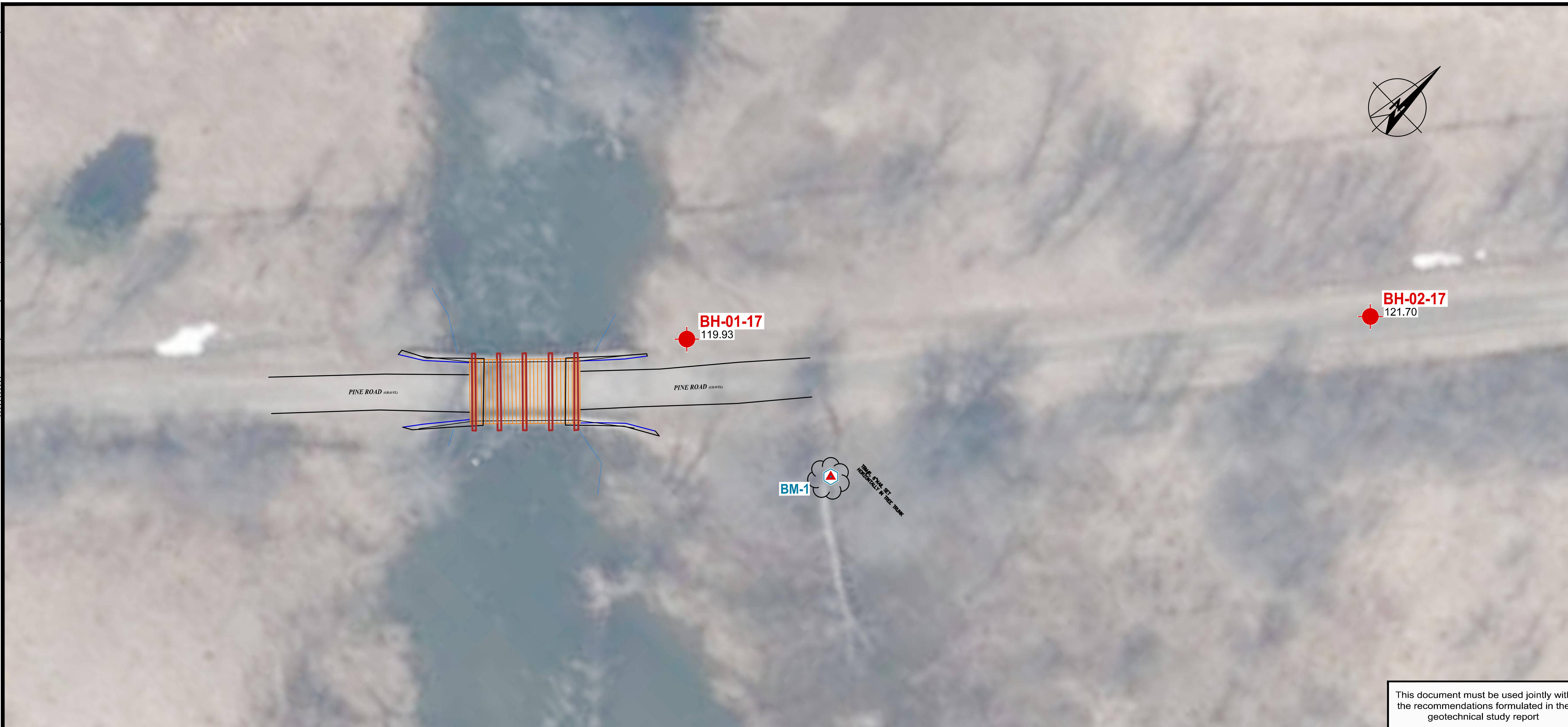
Location: **Trail #50 Bridge in Gatineau Park Near Parking P50, Pine Road, Wakefield, Quebec** File n° : **B-0018588-1**



Symbol	Borehole n°	Sample n°	Depth (m)	W_N	W_L	W_P	I_P	I_L	USCS Class.
○	BH-01-17	SS-11	6.86 - 7.47	64,5	40,0	21,0	19	2,3	CL
⊙	BH-01-17	SS-9	5.33 - 5.94	64,5	49,0	25,0	24	1,6	CL
●	BH-02-17	SS-3	1.22 - 1.83	64,7	85,0	29,0	56	0,6	CH

EQ-09-Ge-69A R.1 04.03.2009

Appendix 4 Borehole location plan



This document must be used jointly with the recommendations formulated in the geotechnical study report

LEGEND:

BH-NN-YY — BOREHOLE-NUMBER-YEAR

BM-1 — BENCHMARK, 6" NAIL SET HORIZONTALLY IN TREE TRUNK

0 1 2 3 5 10 12,5 m

1:250

SURVEY COORDINATES

MTM, NAD83, ZONE 9

BOREHOLE	NORTH (y)	EAST (x)	ELEVATION (m)
BH-01-17	5048089.3	352637.7	119.93
BH-02-17	5048120.9	352666.1	121.70
BM-1	5048089,9	352650,7	120.09

NOTES :
- NCC, PROJET 17059, SHEET 1 OF 1, DATE 24 OCT 2017

Client
National Capital Commission

Project
Soil Investigation
Trail #50 Bridge Replacement
Trail #50 Bridge in Gatineau Park Near Parking P50,
Pine Road, Wakefield, Quebec

Title
Borehole Location Plan

Englobe Corp.
900 de la Carrière Blvd
Suite 100
Gatineau, Quebec
J8Y 6T5
819-778-3143

Discipline: Geotechnical	Prepared by: S. Séguin	Verified by: T. Lampron	
Scale: 1:250	Drawn by: R. Frenette	Approved by: T. Lampron	
Date: 2017-12-20	Figure n°: 1 of 1		
Page setup: 11X17 PAY H	Paper Format: ANSI full bleed B (17,00 x 11,00 pouces)	Register n°:	

Resp.	Project	Wbs	Project/ Disc	Phase/ Type	Elec. ref. / Drawing n°	Rev.
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