National Capital Commission (NCC)

Rehabilitation of a Pedestrian Bridge over Leamy Creek, Gatineau, Québec

Geotechnical Investigation Report

December 2013
O/Ref. : 237-B-0001957-1-GE-R-0002-01

LVM

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Property and Confidentiality

If tests have been done, the results of these tests are valid only for the sample described in the present report.

Testing (either in the field or in laboratory) has been completed by sub-contractors duly qualified according to the purchasing procedure of our quality manual. For more information, please contact your project engineer.

REGISTRE DES RÉVISIONS ET ÉMISSIONS		
Revision No	**Date**	**Description of the modification and/or of the emission**
0A	2012-12-18	Preliminary Report – For comments
0B	2013-01-15	Preliminary Report – For comments
0C	2013-01-24	Preliminary Report – For comments
00	2013-03-13	Final Report
01	2013-12-17	Final Report – English Translation

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INTRODUCTION

National Capital Commission (NCC) awarded a contract to LVM to carry out a geotechnical investigation for the rehabilitation of a pedestrian bridge over Leamy creek in Gatineau, Québec.

The gathered information during this investigation allowed us to obtain the geotechnical properties of soils and the groundwater conditions at the site, in order to formulate geotechnical recommendation for the project, in particular regarding:

- the nature, thickness and the geotechnical properties of the soils;
- the geotechnical bearing capacities;
- the groundwater condition;
- the precaution during excavations;
- the potential of reuse of the excavated material for the backfills;
- the seismic parameters

This investigation was performed in accordance to our proposal identified 12-0045-033 and accepted by the client.

This report presents a site description and the investigation method, as well as a detailed description of the nature and properties of the soils found in the boreholes and the groundwater conditions. Finally, the discussion of the obtained result and geotechnical recommendation for the project are presented in the last section.

The recommendations presented in this report were prepared in accordance with the *Canadian Highway Bridge Design Code* (CAN/CSA-S6-06).

The specific limitations of the investigation, outlined in Appendix 1, should be read jointly with this report.

1 SITE AND PROJECT DESCRIPTION

This project consist in the rehabilitation work the pedestrian bridge over the Leamy creek, located at approximately 130 m from the Fournier boulevard in the Gatineau Park, Québec. This bridge has a length of approximately 56 m. Based on the information provided by the client, this bridge is more than 80 years old. The deck is sitting on three (3) concrete piers, supported of wooden piles.

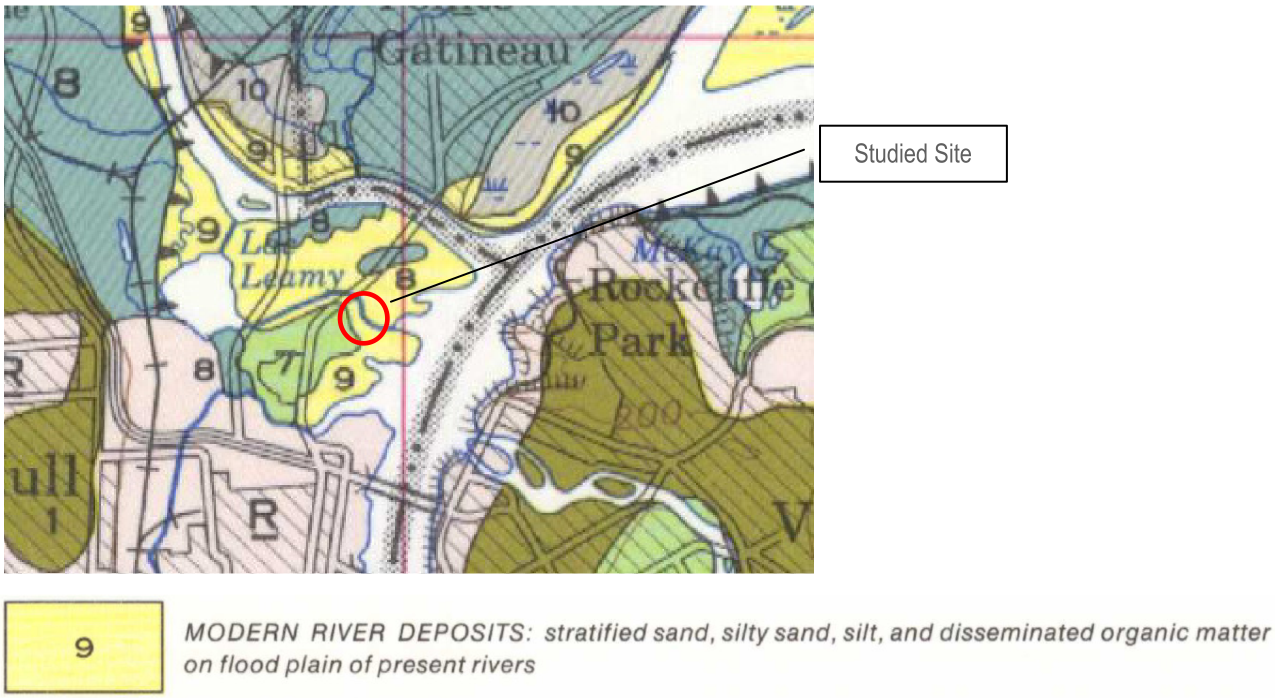
Figure 1 shows the location of the studied site

Figure 1: Location of the Studied Site (source : Bing Maps)

1.1 GEOLOGICAL CONDITION

The regional geological condition of the studied site is illustrated on the geological map "Surficial Materials and Terrain Features" number 1452A, Ottawa-Hull region, produced by the Geological Survey of Canada. We can observe the presence of a modern river deposits.

Figure 2: Surficial Materials and Terrain Features



2 INVESTIGATION PROCEDURES

2.1 FIELDWORK

The fieldwork was performed on December 3rd and 4th 2012. A total of four (4) boreholes were carried and identified from BH-01-12 to BH-04-12.

The four (4) boreholes with sampling were carried out using truck mounted drill of type "CME", on each side of the pedestrian bridge and to the fallowing depth:

Table 1 : Boreholes Depth

Borehole	Depth (m)
BH-01-12	26,92
BH-02-12	8,23
BH-03-12	30,48
BH-04-12	8,23

Soil sampling and Standard Penetration Testing, in accordance with ASTM Standard D 1586-98, were performed with a standard split-spoon sampler of 51 mm outer diameter.

Boreholes BH-01-12 and BH-03-12 were followed by a vane shear test (Nilcon) until a refusal was obtained. Also, a dynamic cone penetration test was performed on BH-01-12 to a depth of 26.92 m, obtaining a refusal on blocks or on probable bedrock, no refusal was obtained on BH-03-12 and the dynamic cone penetration was stopped at 30.48 m.

Two (2) vane shear profile were performed on boreholes BH-01-12 and BH-03-12 using a Nilcon. The undisturbed soil tests were done on a 1 m intervals and the remolded soil test were done on a 2 m intervals. These vane tests were completed to a depth of 10.32 m and 12.97 m form the ground surface. These test allowed us to determine the undrained shear resistance (C_u) and the remolded shear resistance (C_{ur}). Thin walled samples (Shelby) could not be taken due to the stiffness of the clay deposit.

All field work was carried out under the full time supervision of a geotechnical technician from LVM. The subsoil details are presented in the individual borehole logs in Appendix 2.

2.2 LOCATION OF THE BOREHOLES

The site survey to determine the borehole locations was carried out by LVM, based on existing structures on site.

The X and Y coordinate were calculated the plan "09011" dated of June 22nd, 2009. The elevations were also determined from this plan.

The site plan B-0001957-1-GE-D-0001-00, in Appendix 4, shows the position of the boreholes.

2.3 LABORATORY TESTING

All recovered samples were carefully preserved and transported to LVM's laboratory for identification, laboratory testing and classification. All soil samples were examined by a geotechnical engineer and were classified in accordance with the requirements specified in ASTM D2488.

Representative soil samples from the boreholes were submitted to laboratory analyses. Table 2 shows the list of the different analyses performed. The complete laboratory test results are presented in Appendix 3 and are also included on the borehole logs in Appendix 2.

Table 2 : Geotechnical Laboratory Tests Performed

Borehole	Sample	Depth (m)	Grain size analysis (LC 21-040)	Atterberg Limit (BNQ 2501-092)
BH-01-12	SS-2	0.76 – 1.37	✓	
	SS-7	6.10 – 6.71		✓
BH-03-12	SS-1	0.89 – 0.61	✓	
	SS-6	3.81 – 4.42	✓	
	SS-13	9.14 – 9.75		✓

All geotechnical samples recovered from boreholes which were not consumed during laboratory analysis will be stored for a period of six (6) months from the date of completion of the fieldwork; after which, they will be destroyed unless written instructions on the sample storage and/or disposition are received by LVM.

3 NATURE AND PROPERTIES OF SUBSOIL

The following paragraphs present a summary of the different soil layers encountered in the borehole.

The boreholes BH-01-12 and BH-02-12 were completed on the north-east of the bridge near the limit of the deck and the borehole BH-03-12 and BH-04-12 were done on the south-west of the bridge also near the limit of the deck.

Table 3 : Borehole Summary

Abutment	Borehole	Pavement structure Depth (m)		Heterogeneous fill	Silty sand to silt deposit	Silty clay deposit	End of borehole
		Bituminous asphalt	Granular base	Depth (m)	Depth (m)	Depth (m)	Depth (m)
Nord-East	BH-01-12	0.00 – 0.33	0.33 – 1.52	1.52 – 6.10	N/I	6.10 – ≥ 12.32	26.92*
	BH-02-12	0.00 – 0.15	0.15 – 0.76	0.76 – 4.57	N/I	4.57– ≥ 8.22	8.22
South-West	BH-03-12	0.00 – 0.09	0.09 – 0.76	0.76 – 3.81	3.81 – 9.14	9.14 – ≥ 9.75	30.48
	BH-04-12	0.00 – 0.15	0.15 – 0.76	0.76 – 4.57	4.57 – ≥ 8.23	N/I	8.23

* : End of borehole on a refusal

N/I : Not intercepted

3.1 PAVEMENT STRUCTURE

A pavement structure was intercepted on the surface of all the boreholes. This structure is constituted of:

- A layer of 9 cm and 33 cm of asphalt;
- Followed by 0.67 m to 1.19 m of granular foundation constituted of a gravely sand fill with some sill, having a compactness classified as ``compact``.

Two (2) sieve analyses were performed based on representative fill samples. Table 2 shows the results of the analyses which are also presented in Appendix 3 and resumed in Table 4.

Table 4 : Sieve Analysis of the Granular Foundation Fill

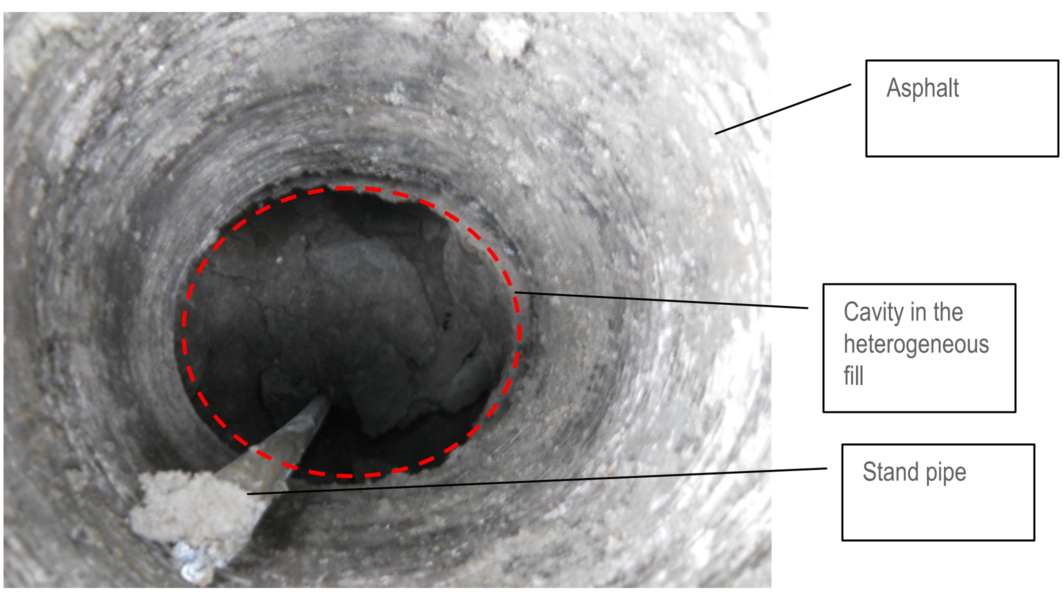
Borehole	Sample	Depth (m)	Gravel > 5 mm (%)	Sand < 5 mm and > 80 µm (%)	Silt and clay < 80 µm (%)	USCS Classification
BH-01-12	SS-2	0.76 – 1.37	43.5	44.0	12.5	SM
BH-03-12	SS-1	0.09 – 0.61	37.7	50.8	11.5	SP-SM

According to the grain size distribution and visual inspection, the tested samples are sandy and gravel with some of silt, classified as SM and SP-SM, according to the Unified Soil Classification System (USCS).

3.2 HETEROGENEOUS FILL

Samples consisting of alternating coarse gravel and a silt and sand were recovered in a unit consisting of a heterogeneous fill. These heterogeneous fill were encountered to depths of 6.10 m and 4.57 m on the northeast shore and up to 3.81 m and 4.57 m on the southwest shore. The lack of material recovery from the augers indicates the presence of cavities. Figure 3 shows a cavity beneath the pavement structure at the borehole BH-01-12.

Figure 3: Cavity Under the Pavement Structure at the Borehole BH-01-12



No laboratory test was performed in this heterogeneious fill due to the low recovery of soil samples.

3.3 SILTY DEPOSIT

A sandy silt to silt deposit was intercepted on the southwest shore at the borehole BH-03-12 and BH-04-12, at a depth of 3.81 m and 4.57 m respectively.

Standard penetration "N" were measured in this deposit with values ranging from 1 to 10, qualifying it compacness of very loose to compact. One (1) representative sample of this deposit was submitted to a grain size analysis. The results of this test are presented in graphic form in annex 3 and are summarized in Table 5.

Table 5 : Results of the Grain Size Analisis in the Silty Deposit

Borehole	Sample	Depth (m)	Gravel > 5 mm (%)	Sand < 5 mm and > 80 µm (%)	Silt and clay < 80 µm (%)	USCS Classification
BH-03-12	SS-6	3.81 – 4.42	0.0	25.3	74.7	ML

According to the grain size distribution and visual inspection, the composition of this deposit is sandy silt, classified as ML, according to the Unified Soil Classification System (USCS).

3.4 SILTY CLAY DEPOSIT

A clayed deposit was intercepted at borehole BH-01-12 to BH-03-12. This deposit was encountered at depths between 3.81 m and 11.25 m, on thickness ranging from 10.49 m to 20.25 m.

A vane shear profile was done in borehole BH-01-12 and BH-03-12. The undrained shear strengths of the clay (C_u) vary between 131 kPa and 166 kPa, describing the consistency of this deposit "very stiff". Also, the values of undrained shear strength of remoulded clay (C_{ur}) vary between 2 kPa and 8 kPa describing the sensitivity of the clay "extra sensitive" to "quick clay".

Laboratory tests conducted on two (2) representative samples taken in this silty clay deposit allowed us to define some geotechnical parameters. The results of these tests are presented in graphic form in Appendix 3 and are summarized in Table 6.

Table 6 : Laboratory Tests on the Silty Clay Deposit

Borehole	Sample	Depth (m)	Wter content « w » (%)	Plasticity limit « w_p » (%)	Liquid limit « w_L» (%)	Plasticity index « I_p» (%)	Liquidity index « I_L»	Classification USCS
BH-01-12	SS-7	6.10 – 6.71	54	28	79	51	0.5	CH
BH-03-12	SS-13	9.14 – 9.75	69	24	68	44	1.0	CH

According to the laboratory results, it is an inorganic clay deposit of high plasticity (Classified as CH, according to the Unified Soil Classification System (USCS).

4 GROUNDWATER

As previously mentioned, in order to determine the groundwater conditions, stand pipes were installed before the withdrawal of the casing from the borehole BH-01-12 and BH-03-12. The level of the river was located at an elevation of 42.61 m, as shown in the plan «Stabilization of the pedestrian bridge over Leamy Creek ", produce by SBA inc. and dated May 2002. Table 7 below shows the measurements taken December 18th, 2012.

Table 7 : Water Measurement on December 18th, 2012

Borehole	Type of installation	Date of reading	Depth of installation (m)	Water level (m)
BH-01-12	Casagrande	2012-12-18	6.41 [el 39.00]	3.87 [el. 41.54]
BH-03-12	Casagrande	2012-12-18	9.41 [el. 36.27]	3.90 [el. 41.78]

The information on groundwater conditions should be interpreted with great caution because the conditions refer only to those observed at the places and dates indicated in this report. It is important to note that the level of water in the soil can be influenced by several factors such as rainfall, snowmelt and changes to the physical environment. Thus, the level of ground water can vary by seasons and over time.

5 GEOTECHNICAL RECOMMENDATIONS

5.1 GENERAL REMARKS

As previously mentioned, the project consists of the rehabilitation of a pedestrian bridge over the Leamy creek in Gatineau, Quebec.

Based on information gathered from the filedwork, the site stratigry can be presented as a pavement structure on the surface, followed by a heterogeneous fill at bridge piers. On the north-east, a very stiff clay deposit was encountered under the heterogeneous fill. On the southwest side, a silty sand deposit with a compactness veriying between “very loose to compact” was intercepted under the fill, followed by a clay deposit at the borehole BH-03-12. The bedrock was not intercepted at a depth of 26.92 m at the borehoole BH-01-12 and 30.48 m at the borehole BH-03-12.

On December 18th 2012, the groundwater levels were between 3.87 m and 3.90 m depth at the location of borehole BH-01-12 and BH-03-12, respectively.

5.1.1 Frost Protection

In order to ensure protection against frost, the top of the piles should be placed at a depth of at least 1.8 m from to the final ground level.

5.2 EXCAVATION AND DEWATERING

Generally, the temporary excavations are the responsibility of the contractor and shall satisfy the minimum requirements of the CSST.

The excavation required in this project will be bone in the heterogeneous fill.

Where there is anough space, the required excavations can be done in open trench and if drainage conditions are favourable. Because these are temporary slopes, the contractor is responsible for their stability as well as the safety of the workers, the construction and the surrounding structures when this security depends on the temporary slope stability. In the event that open trench excavations cannot be achieved, the geotechnical parameters presented in Table 8 (article 5.3) may be used for the design of a system of temporary support of the excavations.

If excavations without retaining systems remain open for extended periods, it is recommended to have daily inspections carried out by specialized geotechnical personnel to detect the risk of slipping and identify measures to correct any anomalies. The resources required (tarps, etc.) must be implemented to protect slopes against erosion due to weather.

It is recommended to avoide parking heavy vehicles near the top of the slope, in a distance lower than the depth of the excavation. It is also recommended to avoid vehicule circulation near the top of the excavations, and, within a distance that is less than the depth of the excavations to minimize the vibrations.

It will also be important to make sure to keep a distance at least equal to the depth of the excavation from the top of the slope and the base of the material piles stored at the site. This condition must be respected at all times unless specific studies are performed for each specific case.

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

Pile driving generates an increase in pore water pressure which could create instability in the abutment fills, depending on their locations. The designer must notify us when the final design will be completed in order to perform slope stability studies with the increase of pore water pressures due to the pile driving.

5.3 TEMPORARY SOIL RETAINING SYSTEM

In the case where non-supported, stable and safe slopes cannot be arranged and adjacent structures limit the workspace, the use of a temporary support system will likely be required.

The temporary support system shall be designed taking into account the stratigraphy of the soil that is in place, the position of the groundwater, as well as existing structures nearby.

Table 8 shows the parameters to consider if the retaining wall is placed so that the slope is fully supported, assuming that the interaction of the soil/support (friction or adhesion) is negligible.

Table 8 : Geotechnical Parameters for the Design of a Temporary Support System

Settings	Value
Heterogeneous Embankment	
Effective cohesion (c')	0 kPa
Effective angle of friction (φ')	32 °
Coefficient of earth pressure at rest (K_o)*	0,47
Coefficient of active pressure (K_a)*	0,31
Coefficient of pasive pressure (K_p)*	3,25
Clay deposit (Calculation parameters in the long-term)	
Effective cohesion (c')	5 kPa
Effective angle of friction (φ')	28°
Coefficient of earth pressure at rest (K_o)*	0,53
Coefficient of active pressure (K_a)*	0,36
Coefficient of pasive pressure (K_p)*	2,77
Clay deposit (Calculation parameters at short-term)	
Undrained cohesion (c_u)	Voir rapport de forage
Angle of undrained internal friction (φ_u)	0°
Silt deposit	
Effective cohesion (c')	0 kPa
Effective angle of friction (φ')	28°
Coefficient of earth pressure at rest (K_o)*	0,53
Coefficient of active pressure (K_a)*	0,36
Coefficient of pasive pressure (K_p)*	2,77
General parameters	
Wet unit weight of heterogeneous fill (γ)	19 kN/m^3
Effective unit weight of heterogeneous fill (γ')	9 kN/m^3
Wet unit weight of the clayed deposit (γ)	16 kN/m^3
Effective unit weight of the clayed deposit (γ')	6 kN/m^3
Wet unit weight of the silt deposit (γ)	17 kN/m^3
Effective unit weight of the silt deposit (γ')	7 kN/m^3
** Case of horizontal surface and vertical walls.*	

All values shown in the above table are taken from the literature.

5.4 SHALLOW FOUNDATIONS

In boreholes drilled at the site of the existing abutments, the soil deposit was intercepted at 6.10 m at borehole BH-01-12 on the north-west side, and at 3.81 m at the borehole BH-03-12. The overlying heterogeneous fill is not able to support the foundation, it is recommended to install the foundations on the natural deposit. Therefore, conventional foundations with strip footings type may be used.

The following recommendations are given according to the directive of the *Canadian Highway Bridge Design Code* (CAN/CSA-S6-06, November 2006). This code requires that the calculation of shallow foundations be carried out according to the limit states method. The limit states are divided into two (2) goups:

- The ultimate limit state (ULS);
- The serviceability limit states (SLS).

The ultimate limit state ULS focuses mainly on structural collapse mechanisms; thus, focusing on security, whereas the serviceability limit states SLS focuses on the mechanisms that limit or prevent the intended use of the structure such as the total and differential settlements.

5.4.1 Ultimate Limit State (ULS)

In order to determine the ultimate limit state (ULS) for the bearing capacity in the till deposit, the designer must take into account the inclination of the resultant geometry of the foundation and the eccentricity of the load. The following parameters are provided to the designers and can be used for the calculation of the bearing capacity ultimate.

The formula to use is the following:

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

ULS = Ultimate Limit State

q' = pressure at level of footings

c' = Effective cohesion = 0 for the granular soil

In the case where there is an eccentric load, the width of the base must be modified to take into account the eccentricity, with an effective width B' and a length *L'* where:

$B' = B - 2e_B$, but inferior to L'

$L' = L - 2e_L$

e: the eccentricity of the load

S_c, S_q, S_γ are the coefficients of form allowing taking into account the geometry of the foundation:

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

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

I_c, I_q, I_γ are the coefficients of the tilt allowing to take into account the inclination of the load:

$I_c = I_q = (1\text{-}\ \delta/90°)^2$

$I_\gamma = (1\text{-}\ \delta/\phi')^2$

δ: is the angle of the resultant force in relation to the vertical

The geotechnical parameters recommended for the calculations at the ULS are those presented in Table 9.

Table 9: Geotechnical Parameters for the Calculation of the ULS Resistance

Parameters	North-East Abutment Silty Clay Deposit	South-West Abutment Sandy Silt Deposit
Effective cohesion (*c'*) (kPa)	5 kPa	0 kPa
Wet unit weight	16 kN/m³	17 kN/m³
Effective unit weight	6 kN/m³	7 kN/m³
Effective angle of friction (ϕ')	28°	30°
Bearing capacity factor for cohesion (N_c)	26	30
Bearing capacity factor for earth pressure (N_q)	15	18
Bearing capacity factor for the soil weight (N_γ)	11	16
Width of the footing (*B*)	To be defined by the designer	To be defined by the designer
Depth of foundation (*D*)	To be defined by the designer	To be defined by the designer

To obtain the factored ultimate limit state, the *National Building Code* 2010 recommend to apply a coefficient of 0.5 to the value of the ultimate limit state.

5.4.2 Serviceability Limit States (SLS)

As mentioned in the *Canadian Highway Bridge Design Code*, the geotechnical resistance at the SLS must make use of geotechnical parameters adapted to the site conditions while taking into account both the differential and total settlement in the short and long term.

A geotechnical resistance value for SLS of 100 kPa was calcuylated in the gray silty clay deposit with a stiff consistency, at the north-east abutment where the elevation of the foundation is 39.31 m. A serviceability limit state of 60 kPa was obtained in the sandy silt deposit, for the south-west abutment, with an elevation of foundation of 41.87 m. This value takes into account a width B of 2 m and a depth of foundation D of 6.10 m and 3.81 m respectively. The geotechnical resistance at the SLS includes a safety factor minimum of 3 against the shear failure and allows a total settlement lower than 25 mm and a differential setlement smaller than 19 mm.

The values of the movement presented above assume, however, that the seating of the foundation will be free of any mud and of any disturbed soil before proceeding to pouring concrete for the foundations.

5.5 DEEP FOUNDATIONS

According to the data received from the designer, the use of deep foundations is preferred for the reconstruction of the bridge. The following recommendations are given for the design of deep foundations.

5.5.1 General Comments for Driven Piles

Various types of piles can be considered (steel H-sections piles or tubular steel pipe piles with closed ends). The piles can be supported in the silty clay deposit. The design and installation of the piles must be carried out in accordance with the requirements of section 15.3 of the CCDG as well as the requirements of the Canadian Highway Bridge Design Code, CSA-S6-06.

While making the choice of the pile, it is recommended to substract 1.5 mm of the thickness of the pile walls, to compensate for the corrosion. In the case of tubular piles, it is recommended to subtract the 1.5 mm wall thickness if the tubes are not filled with concrete and 1.0 mm if the piles are filled with concrete.

In every case, the piles must be made using steel according to the requirements CSA G40.21 and having a structural quality and a minimum thickness sufficient for transmitting enough forces to penetrate soils containing boulders and blocks. This will also take into account additional constraint and that were not considered during the design such as the constraints imposed by the bending of the pile, the eccentricity of the hammer during pile driving and the driving force on the tip of the pile transferred to to the force required for the penetration in the soil.

It is strongly recommended that the tip of the piles is protected with a driven protector to prevent damage to the pile where boulders and blocks are encountered.

The rigidity of the piles can be increased by filling them with concrete after the installation.

Difficulties may occur during pile driving the pipe into the clay. Bouncing piles is common in sensitive clays. If this occurs during driving, it would be required to prepare a driving sequence to allow time for porewater pressures to dissipate around the piles. The pile can also be filled with water to increase the weight. This problem can also be solved by using tubular piles with open toe. This approach has also the advantage of reducing the porewater pressures generated by pile driving. This solution is highly recommended for friction tubular piles in sensitive clayed soils such the ones found on this site.

5.5.2 Geotechnical Ultimate Axial Resistance (Compession)

The method described in the Canadian Foundation Engineering Manual (CFEM), 4th Edition, section 18.2.1, is recommended to determine the geotechnical axial capacity at ultimate limit states (R_u).

The axial capacity at ultimate limit states (R_u) of a single pile can be estimated by summing the friction resistance along the pile (q_s) and adding the toe resistance (q_p), where C is the circumference of the pile, At is the tip section and Wp is the weight of the pile.

$$R_u = \underbrace{\left(\sum_{z=0}^{L} C \times q_s \times \Delta z \right)}_{\text{friction resistance}} + A_t \times q_p - Wp \qquad où \quad \begin{cases} q_s = \beta \times \sigma'_v \\ q_p = N_t \times \sigma'_{tip} \end{cases}$$

The β and N_t are dimensionless parameter which depend on the type of soil considered and σ'_v et σ'_{tip} are effective soil constraints at the pile depth and at the toe depth.

Table 10 present the recommended value for the evaluation of the constraints.

Table 10 : Geotechnical Ultimate Axial Resistance for a Single Pile – Parameters

Parameters	Recommended Value for a Driven Pile
Parameter β	
Silty clay deposit	0.3
Silt deposit	0.3
Parameter N_t	
Clay deposit	40
Genral parameters	
Wet unit weight of the clayed deposit (γ)	16 kN/m^3
Effective unit weight of the clayed deposit (γ')	6 kN/m^3
Wet unit weight of the silt deposit (γ)	17 kN/m^3
Effective unit weight of the silt deposit (γ')	7 kN/m^3

The geotechnical ultimate axial resistance should be factored. A value of 0.4 is recommended in the Canadian Highway Bridge Design Code, CSA-S6-06 for the compressive strength. If tests are performed in situ, the resistance factor may be increased to 0.6. Otherwise, it is recommended that the resistance factor used is at most 0.4. It is strongly recommended that load tests on piles were performed to verifiy the bearing capacity in relation to the refusal parameters used (ASTM D- 1143 "Piles Under Static Axial Compressive Load"). Alternatively, the use of a pile driving analyzer could be used for a minimum of 10% of the driven piles.

Also, it is strongly recommended that the load tests are performed in the beginning of the poile drving to establish the refusal criteria based on the bearing capacity required, and to conduct other tests during and at the end.

To mobilize sufficient axial geotechnical resistance, piles should be driven until an adequate support capacity. The refusal criteria must be established using the analysis by the equation of waves (Wave Equation Analysis).

5.5.3 Pull Out Resistance of the Piles

To estimate the pullout resistance of a pile, Chapter 18.2.6 of the Canadian Foundation Engineering Manual, 2006 recommends using 75% of the mobilized friction resistance along the pile shaft. Therefore, to calculate the peel strength in the case of piles, the following equation may be used:

$$R_{pullout} = 0,75\left(\sum_{z=0}^{L} C \times q_s \times \Delta z\right) + Wp \qquad were\ \ q_s = \beta \times \sigma'_v$$

β = see Table 10

C = perimeter of the pile (m)

σ'_v = Average effective vertical stress at a depth z (kPa).

Δz = thickness of the considered soil layer (m)

W_p = Weight of the pile (kN)

If the piles are filled with concrete, it is possible to include it in the weight. To calculate the weight of the concrete, we recommend using an effective unit weight of 14 kN/m^3.

The Canadian Highway Bridge Design Code, CSA-S6-06 recommends a coefficient of 0.3 for the ultimate geotechnical resistance for pull out of the piles.

The pull out geotechnical resistance for a pile group is the lower of these two (2) values:

- the summation of the individual pull out resistance of each pile ;
- the summation of the pull out resistance on the perimeter of the pile group plus weight of the soil and pile inside this perimeter.

5.5.4 Geotechnical Lateral Resistance

Vertical piles subjected to lateral loads tend to deform and this deformation is related with the support of the surrounding soil. The behavior of the Foundation under such conditions depends essentially on the stiffness of the pile and the soil strength. The lateral support of the vertical piles can be limiterd by three (3) factors:

The load exceeds the horizontal geotechnical resistance, which results in significant horizontal movements and a foundation failure.

The shear moment exceed the strength of the pile itself, which results in a rupture of the pile.

The movements at the tip of the piles are too high compared to the tolerances of the structure.

Each of these failure modes must be considered in the design. The designer should refer to Article 6.8.7 of the Canadian Highway Bridge Design Code to evaluate the factored lateral geotechnical resistance. The Table 6.6.2.1 of the code recommends a lateral passive pressure coefficient of 0.5.

To determine the lateral geotechnical resistance of the piles, the geotechnical and geology service of the Ministry of Transport of Quebec recommends using the Broms method, which is detailed in Appendices 1 and 2 of the special specification requirement 110 prepared by the geotechnical and geology service of the Ministry of Transport of Quebec. The geotechnical soil parameters used in the calculation of the lateral geotechnical resistance are shown in Table 11.

Table 11 : Geotechnical Parameter for the Soil

Geotechnical parameter	Silty clay deposit	Silt deposit
Type of soil	Cohesive	Granular
Horizontal reaction coefficient n_h (N/m³)	$67C_u/b$	$1\,100 \times 10^3$
Ajsuted horizontal reaction coefficient n_h (N/m³)	$33\ C_u/b$	275×10^3
Rankine passive earth pressure K_p	2,77	2,77
Satureted unit weight (γ) (kN/m³)	6	7
Cu : Undrained shaer resistance résistance of undisturbed clay (see borehole log)		
B : width or diameter of the pile (m)		

A correction resistance factor must be applied to a group of pile to consider the spacing between them, as shown in Table 12 below.

Table 12 : Correction Factor

Spacing between the piles	Reduction factor for a group of piles in a granular soil *	Reduction factor for a group of piles in a cohesive soil †
8b	---	1.0
6b	0.7	0.65
4b	0.6	0.5
3b	0.5	0.4
Note : b = Diameter of the piles		

If the lateral geotechnical resistance measured is insufficient, the use of drilled or inclined piles may be required.

In the case of inclined piles, the geotechnical and geology service of the Ministry of Transport of Quebec recommends that the lateral capacity of inclined piles be adjusted according to their inclination to the vertical and the direction of the load.

5.6 ABUTMENTS

5.6.1 Earth Pressure

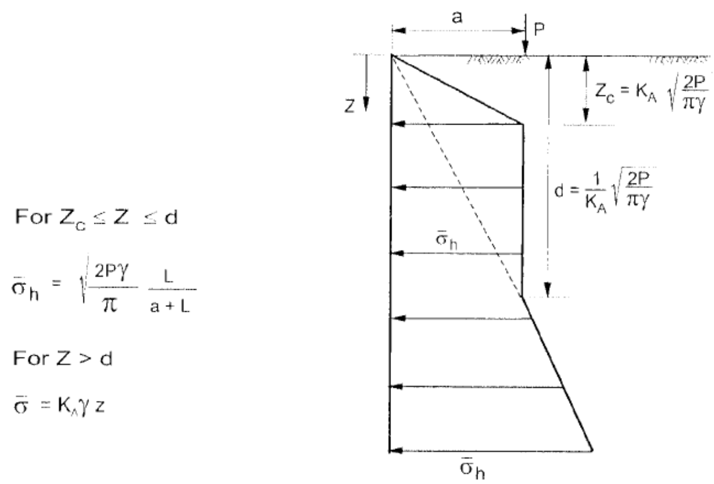
The abutments walls will be subjected to the earth pressure caused by the backfill behind them. For this purpose, it is necessary to refer to Section 6.9 of the Canadian Highway Bridge Design Code.

In general, if the fill material used must be compacted to a distance less than 3 m of the walls, it is necessary to use the stress distribution shown in Figure 4 for the calculation of the earth pressure.

* Oteo, C.S., "Displacements of a Vertical Pile Group Subjected to Lateral Loads", Proceedings 5th European Conference of Soil Mechanics and Foundation Engineering, Madrid, Vol. 1, 1972, pp. 397-405.

† Prakash, S. and Saran D., "Behavior of Laterally Loaded Piles in Cohesive Soil" Proceedings 3rd Asian Regional Conference on Soil Mechanics and Foundation Engineering, Haifa (Israel), 1967, pp. 235-238.

Figure 4 : Stress Distribution due to the Compaction of the Backfill



$$P \text{ (roller load)} = \frac{\text{dead weight of roller + centrifugal force}}{\text{weight of roller}}$$

a = distance of roller from wall

L = length of roller

The properties of the granular material used for backfill behind the wall are indicated in Table 13.

Table 13 : Geotechnical Parameters for Lateral Earth Pressure

Parameter	Granular material MG-112, CG-14 OR MG-20 (1)
Wet unit weight (γ)	20 kN/m³
Angle of internal friction (ϕ')	36°
Coefficient of earth pressure at rest (K_o)	0.41*
Coefficient of active pressure (K_a)	0.26*
Coefficient of pasive pressure (K_p)	3.85*

* *Case of horizontal surface and vertical walls.*

(1) Compacted to 95% of the maximum dry density of the material , as determined in (NQ 2501-255)

The coefficient K_a is used for structure unsupported at the top while the coefficient K_o is used for structures supported at the top.

5.7 REUSE OF THE EXCAVATED MATERIAL FOR THE BACKFILL

Within the approach of the projected structure, the transitions and backfilling must be made according to the requirements defined in the standard drawing 1-021 "Volume of II Standards of Road Works" of the Ministry of Transportation of Quebec.

The excavated backfill can not be reused because of their heterogeneity. The cohesive excavated materials can not be reused as backfill given the high percentage of fine particules (<80 microns).

5.8 SITE COEFICIENT FOR THE CALCULATION OF DINAMIC LOADS

Based on the information gathered during this investigation, the soil to consider for evaluating the sismic site coefficient is a soil of type III, according to the descriptions provided in section 4.4.6 of the Code on bridge design road. According to Table 4.4 of the Canadian Highway Bridge Design Code, a coefficient of 1.5 should be considered for a soil of Type III.

Appendix 1 Limitation of the Investigation

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 LVM, allowing the validity of the recommendations presented to be verified. Complementary site or laboratory work could ultimately be required.

This report cannot be reproduced, in whole or in part, without the authorization of LVM.

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, LVM 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, LVM 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 Explanation Notes on the Boring Log, Boring 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.

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B-0001957-1

Client :

National Capital Commission

BOREHOLE REPORT

File n°:	**B-0001957-1**
Borehole n°:	**BH-01-12**
Date:	**2012-12-03**

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

Location: **Route Verte 1, Abutment Northeast, Gatineau, Quebec**

Coordinates (m): North 5034743.0 (Y) East 366812.0 (X) Elevation **45.41 (Z)**

Bedrock: m End depth: 26.92 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	LVM 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

Undrained shear strength — Field / Laboratory

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

STRATIGRAPHY / SAMPLES / FIELD AND LABORATORY TESTS

ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION	TYPE AND NUMBER	RECOVERY %	Blows/150mm	"N" or RQD	RESULTS
45.41 / 0.00	Asphalt					
45.08 / 0.33	***Pavement Structure:*** Sand and gravel with some silt	SS-1	100	50 /5 cm	R	
		SS-2	42	10-8 5-5	13	GS
43.89 / 1.52	***Heterogeneous Fill:*** Grey sand	SS-3	20	50	R	
43.12 / 2.29	Brown silty sand with some gravel	SS-4	8	10-5 3-5	8	
		SS-5	38	7-20 26-8	46	
40.84 / 4.57	Brown sandy gravel, saturated	SS-6	33	10-14 5-7	19	
39.31 / 6.10	***Clay Deposit :*** Grey clay, very stiff consistency	SS-7	100	2-3 4-3	7	
38.70 / 6.71	End of sampling at 6.71 m					
38.09 / 7.32	Beginning of the field vane testing at 7.32 m					C_U= 138 kPa
37.79						C_U= 140 kPa, C_{UR}= 2 kPa

WATER LEVEL (m) / DATE: el. 41.54 m 2012-12-18

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

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION — 20 40 60 80 100 120 140 160 180

Vertical Scale = 1 : 75

Remarks:

Borehole type: **Auger** Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.** Approved by: **T. Lampron, Jr. Eng.** 2013-12-16

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BOREHOLE REPORT

Client : **National Capital Commission**

File n°:	**B-0001957-1**
Borehole n°:	**BH-01-12**
Date:	**2012-12-03**

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

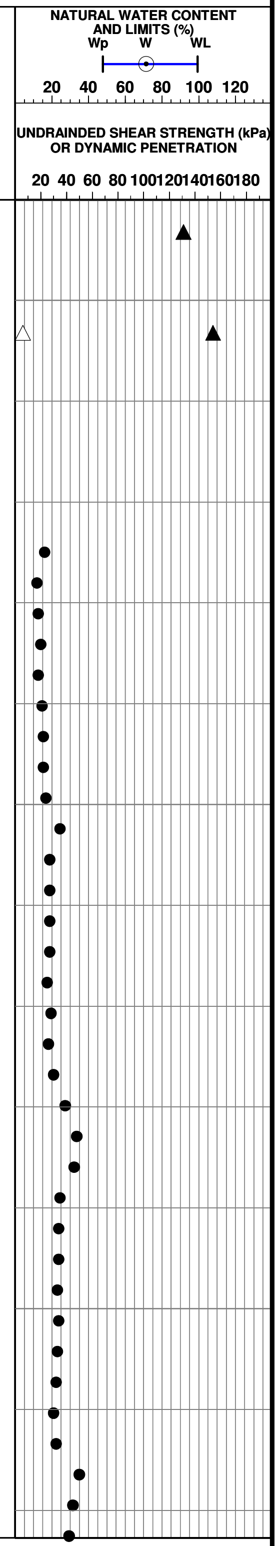
Location: **Route Verte 1, Abutment Northeast, Gatineau, Quebec**

Coordinates (m):		
	North	5034743.0 (Y)
	East	366812.0 (X)
	Elevation	**45.41 (Z)**
Bedrock: m	End depth:	26.92 m

Vertical Scale = 1 : 75

DEPTH - ft	DEPTH - m	ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION	RESULTS
30–31				C_U= 131 kPa
34		10.32	End of the field vane test at 10.32 m	C_U= 154 kPa C_{UR}= 6 kPa
41		32.91 12.50	Beginning of the dynamic penetration test at 12.49 m	N_c= 23
42				N_c= 17
43	13			N_c= 18
44				N_c= 20
45				N_c= 18
46	14			N_c= 21
47				N_c= 22
48				N_c= 22
49	15			N_c= 24
50				N_c= 35
51				N_c= 27
52	16			N_c= 27
53				N_c= 27
54				N_c= 27
55				N_c= 25
56	17			N_c= 28
57				N_c= 26
58				N_c= 30
59	18			N_c= 39
60				N_c= 48
61				N_c= 46
62	19			N_c= 35
63				N_c= 34
64				N_c= 34
65				N_c= 33
66	20			N_c= 34
67				N_c= 33
68				N_c= 32
69	21			N_c= 30
70				N_c= 32
71				N_c= 50
72	22			N_c= 45

Column headings: STRATIGRAPHY (ELEVATION - m / DEPTH - m; SOIL OR BEDROCK DESCRIPTION; SYMBOLS); WATER LEVEL (m) / DATE; SAMPLES (TYPE AND NUMBER; SUB-SAMPLE; CONDITION; SIZE; RECOVERY %; Blows/150mm; "N" or RQD; Organo. Exam: Odor, Visual); FIELD AND LABORATORY TESTS (RESULTS; NATURAL WATER CONTENT AND LIMITS (%) Wp W WL; UNDRAINED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION).



Remarks:

Borehole type: **Auger** Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.** Approved by: **T. Lampron, Jr. Eng.** 2013-12-16

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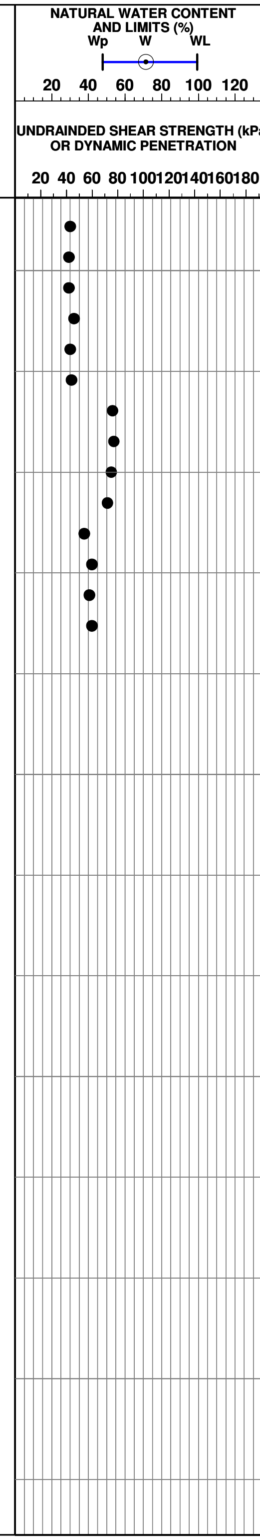
B-0001957-1

LVM	Client : **National Capital Commission**	**BOREHOLE REPORT**
		File n°: **B-0001957-1**
		Borehole n°: **BH-01-12**
		Date: **2012-12-03**

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**	Coordinates (m): North 5034743.0 (Y)
	East 366812.0 (X)
Location: **Route Verte 1, Abutment Northeast, Gatineau, Quebec**	Elevation **45.41 (Z)**
	Bedrock: m End depth: 26.92 m

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
															N_c= 42
74															N_c= 43
75	23														N_c= 42
76															N_c= 42
77															N_c= 46
78															N_c= 43
79	24														N_c= 44
80															N_c= 76
81															N_c= 77
82	25														N_c= 75
83															N_c= 72
84															N_c= 54
85	26														N_c= 60
86															N_c= 58
87															N_c= 60
88		18.49													N_c= Refusal
89	27	26.92	End of the dynamic penetration test at 26.92 m after obtaining a refusal on dense soil or probable rock												
90															
91															
92	28														
93															
94															
95	29														
96															
97															
98	30														
99															
100															
101															
102	31														
103															
104															
105	32														
106															
107															
108	33														
109															
110															
111															
112	34														
113															
114															
115	35														
116															

FIELD AND LABORATORY TESTS



Vertical Scale = 1 : 75

Remarks:

Borehole type: **Auger** Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.**	Approved by: **T. Lampron, Jr. Eng.**	2013-12-16	Page: 3 of 3

BOREHOLE REPORT

LVM

Client : **National Capital Commission**

File n°:	**B-0001957-1**
Borehole n°:	**BH-02-12**
Date:	**2012-12-03**

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

Location: **Route Verte 1, Abutment Northeast, Gatineau, Quebec**

Coordinates (m): North 5034742.0 (Y); East 366816.0 (X); Elevation **45.82 (Z)**

Bedrock: m End depth: 8.23 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	LVM 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

Undrained shear strength (Field / Laboratory)

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

STRATIGRAPHY / SAMPLES / FIELD AND LABORATORY TESTS

ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION	TYPE AND NUMBER	RECOVERY %	Blows/150mm	"N" or RQD
45.82 / 0.00	Asphalt				
45.67 / 0.15	***Pavement Structure:*** Grey sandy gravel with traces of silt	SS-1	33	11-8 7-7	15
45.06 / 0.76	***Heterogeneous Fill:*** Grey gravelly sand with traces of silt	SS-2	42	4-3 4-8	7
		SS-3	25	3-4 3-3	7
		SS-4	17	2-4 5-3	9
42.77 / 3.05	Sand with some gravel and traces of silt and organic matter	SS-5	21	2-3 4-4	7
		SS-6	17	4-9 6-3	15
41.25 / 4.57	Brown sandy gravel, saturated	SS-7	29	1-1 1-1	2
40.49 / 5.33	***Clay Deposit:*** Grey silty clay, slightly damp	SS-8	100	2-3 3-3	6
		SS-9	100	2-2 2-2	4
		SS-10	100	1-1 1-2	2
37.59 / 8.23	End of borehole				

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

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION — 20 40 60 80 100 120 140 160 180

Remarks:

Borehole type: **Auger** Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.** Approved by: **T. Lampron, Jr. Eng.** 2013-12-16

LVM

Client :

National Capital Commission

BOREHOLE REPORT

File n°: **B-0001957-1**
Borehole n°: **BH-03-12**
Date: **2012-12-04**

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

Location: **Route Verte 1, Abutment Southeast, Gatineau, Quebec**

Coordinates (m): North 5034724.0 (Y); East 366753.0 (X); Elevation **45.68 (Z)**

Bedrock: m End depth: 30.48 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	LVM 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

Undrained shear strength (Field / Laboratory)

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

STRATIGRAPHY

ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION
45.68 / 0.00	Asphalt
45.59 / 0.09	***Pavement Structure :*** Sand and gravel with some silt
44.92 / 0.76	***Heterogeneous Fill:*** Silty sand with traces of clay and some wood
44.16 / 1.52	Grey sand with some silt and some gravel
41.87 / 3.81	***Silty Sand to Silt Deposit :*** Sandy silt, brown, saturated
40.35 / 5.33	Brown silty fine sand saturated
39.58 / 6.10	Grey micaceous silty fine sand, saturated

WATER LEVEL (m) / DATE: el. 41.78 m 2012-12-18

SAMPLES / FIELD AND LABORATORY TESTS

TYPE AND NUMBER	RECOVERY %	Blows/150mm	"N" or RQD	RESULTS
SS-1	48	6-9 4	13	GS
SS-2	21	3-4 6-7	10	
SS-3	12	2-2 1-9	3	
SS-4	21	2-4 6-19	10	
SS-5	38	13-11 16-49	27	
SS-6	71	2-2 3-4	5	GS
SS-7	79	2-3 2-3	5	L W = 54.0
SS-8	75	4-5 5-4	10	
SS-9	71	1-2 2-2	4	
SS-10	71	2-2 3-2	5	
SS-11	83	1-1 1-1	2	
SS-12	71	0-0 2-1	2	

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

UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION — 20 40 60 80 100 120 140 160 180

Remarks:

Borehole type: **Auger**

Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.** | Approved by: **T. Lampron, Jr. Eng.** | 2013-12-16 |

LVM

Client : **National Capital Commission**

BOREHOLE REPORT

File n°:	**B-0001957-1**
Borehole n°:	**BH-03-12**
Date:	**2012-12-04**

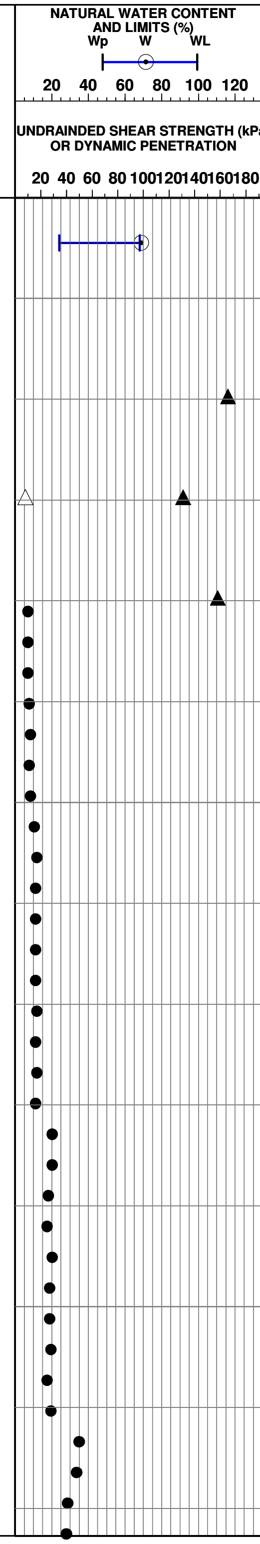
Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

Location: **Route Verte 1, Abutment Southeast, Gatineau, Quebec**

Coordinates (m):		
	North	5034724.0 (Y)
	East	366753.0 (X)
	Elevation	**45.68 (Z)**
Bedrock: m	End depth:	30.48 m

Vertical Scale = 1 : 75

DEPTH - ft	DEPTH - m	ELEVATION - m / DEPTH - m	STRATIGRAPHY: SOIL OR BEDROCK DESCRIPTION	SAMPLES: TYPE AND NUMBER	SUB-SAMPLE	CONDITION	SIZE	RECOVERY %	Blows/150mm	"N" or RQD	Organo. Exam: Odor	Organo. Exam: Visual	FIELD AND LABORATORY TESTS: RESULTS
30–32		9.14	***Clay Deposit:*** Grey silty clay with traces of sand very stiff consistency	SS-13				50	0-0 1-1	1			L W = 69.0
32–33	10	35.93 9.75	End of sampling at 9.75 m										
36	11	34.71 10.97	Beginning of the field vane testing at 10.97 m										C_U= 166 kPa
39	12												C_U= 131 kPa C_{UR}= 8 kPa
42–43	13	32.71 12.97	End of the field vane test at 12.97 m										C_U= 158 kPa N_C= 10
44		32.57 13.11	Beginning of the dynamic penetration test at 13.11 m										N_C= 10
45													N_C= 10
46	14												N_C= 11
47													N_C= 12
48													N_C= 11
49	15												N_C= 12
50													N_C= 15
51													N_C= 17
52	16												N_C= 16
53													N_C= 16
54													N_C= 16
55													N_C= 16
56	17												N_C= 17
57													N_C= 16
58													N_C= 17
59	18												N_C= 16
60													N_C= 29
61													N_C= 29
62	19												N_C= 26
63													N_C= 25
64													N_C= 29
65	20												N_C= 27
66													N_C= 27
67													N_C= 28
68													N_C= 25
69	21												N_C= 28
70													N_C= 50
71													N_C= 48
72	22												N_C= 41



Remarks:

Borehole type: **Auger**

Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.**	Approved by: **T. Lampron, Jr. Eng.**	2013-12-16	Page: 2 of 3

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Client : **National Capital Commission**

BOREHOLE REPORT

File n°:	**B-0001957-1**
Borehole n°:	**BH-03-12**
Date:	**2012-12-04**

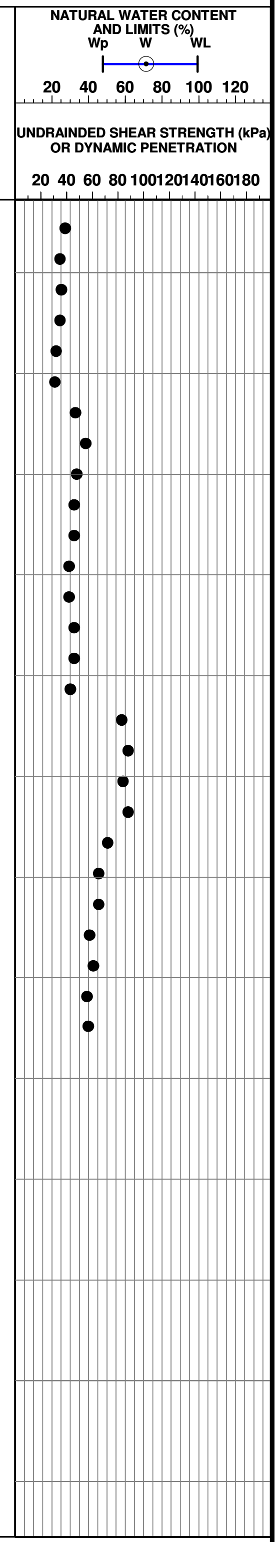
Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

Location: **Route Verte 1, Abutment Southeast, Gatineau, Quebec**

Coordinates (m):		
	North	5034724.0 (Y)
	East	366753.0 (X)
	Elevation	**45.68 (Z)**
Bedrock: m	End depth:	30.48 m

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
															N_c = 40
74															N_c = 39
75	23														N_c = 35
76															N_c = 36
77															N_c = 35
78															N_c = 32
79	24														N_c = 31
80															N_c = 47
81															N_c = 55
82	25														N_c = 48
83															N_c = 46
84															N_c = 46
85	26														N_c = 42
86															N_c = 42
87															N_c = 46
88	27														N_c = 46
89															N_c = 43
90															N_c = 83
91															N_c = 88
92	28														N_c = 84
93															N_c = 88
94															N_c = 72
95	29														N_c = 65
96															N_c = 65
97															N_c = 58
98	30														N_c = 61
99		15.20													N_c = 56
100		30.48	End of the dynamic penetration test without refusal												N_c = 57
101															
102	31														
103															
104															
105	32														
106															
107															
108	33														
109															
110															
111															
112	34														
113															
114															
115	35														
116															

FIELD AND LABORATORY TESTS



Remarks:

Borehole type: **Auger**

Boring equipment: **CME-55**

Prepared by: **S. Séguin, tech.** | Approved by: **T. Lampron, Jr. Eng.** | 2013-12-16 | Page: 3 of 3

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Vertical Scale = 1 : 75

EQ-09-Ge-66A R.1 04.03.2009

BOREHOLE REPORT

Client : **National Capital Commission**

File n°: **B-0001957-1**
Borehole n°: **BH-04-12**
Date: **2012-12-04**

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek**

Location: **Route Verte 1, Abutment Southeast, Gatineau, Quebec**

Coordinates (m): North 5034723.0 (Y); East 366758.0 (X); Elevation **0.00 (Z)**

Bedrock: m End depth: 8.23 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	LVM 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

Undrained shear strength (Field / Laboratory)

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

ELEVATION - m / DEPTH - m	SOIL OR BEDROCK DESCRIPTION	TYPE AND NUMBER	RECOVERY %	Blows/150mm	"N" or RQD
0.00 / 0.00	Asphalt				
-0.15 / 0.15	***Pavement Structure:*** Sandy gravel with some silt, with grey-brown lump of clay	SS-1	67	6-8 10	18
-0.76 / 0.76	***Heterogeneous Fill:*** Sand grey with some silt and trace of clay	SS-2	38	2-4 4-3	8
		SS-3	17	4-8 5-2	13
-2.29 / 2.29	Sand and silt with traces of gravel, wet	SS-4	21	6-9 10-14	19
-3.05 / 3.05	Coarse gravel with some sand and silt	SS-5	29	3-9 10-14	19
		SS-6	36	17-10 50 /5 cm	60
-4.57 / 4.57	***Silty Deposit :*** Silt and sand, brown saturated	SS-7	79	2-2 2-2	4
		SS-8	58	2-1 2-2	3
		SS-9		1-1 2-3	3
-8.23 / 8.23	End of the borehole				

FIELD AND LABORATORY TESTS: RESULTS; NATURAL WATER CONTENT AND LIMITS (%) Wp W WL: 20 40 60 80 100 120; UNDRAINDED SHEAR STRENGTH (kPa) OR DYNAMIC PENETRATION: 20 40 60 80 100 120 140 160 180

Remarks:

Borehole type: **Auger**
Boring equipment: **CME-55**

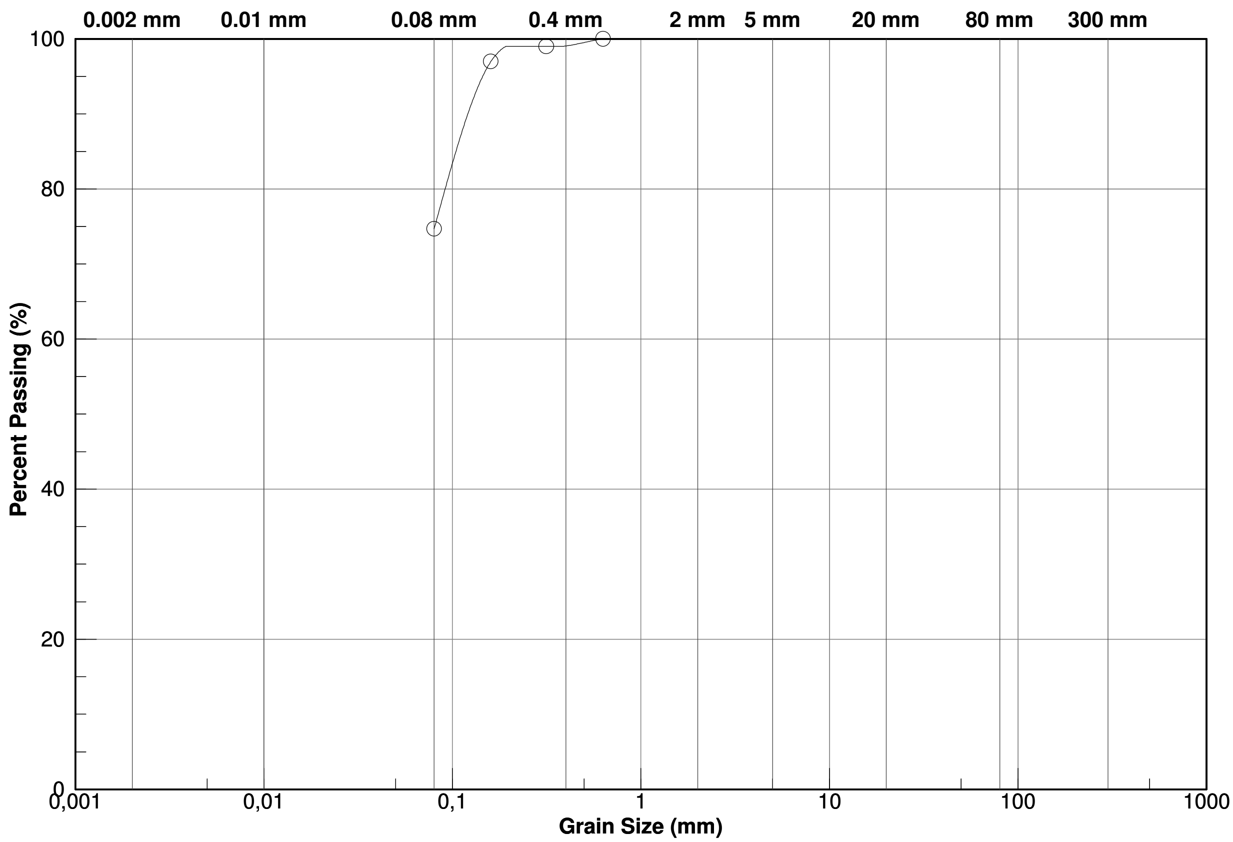
Prepared by: **S. Séguin, tech.**
Approved by: **T. Lampron, Jr. Eng.**
2013-12-16

Appendix 3 Laboratory Tests

GRAIN-SIZE ANALYSIS

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek** Figure n° : **1**

Location: **Route Verte 1, Abutment Southeast, Gatineau, Quebec** File n° : **B-0001957-1**



CLAY	SILT	SAND			GRAVEL		COBBLE	BOULDER
		FINE	MEDIUM	COARSE	FINE	COARSE		

Col. symboles	Borehole n°	Sample n°	Depth (m)	Description	USCS class. (ASTM D-2487)
○	BH-03-12	SS-6	3.81 - 4.42	Sandy silt	ML

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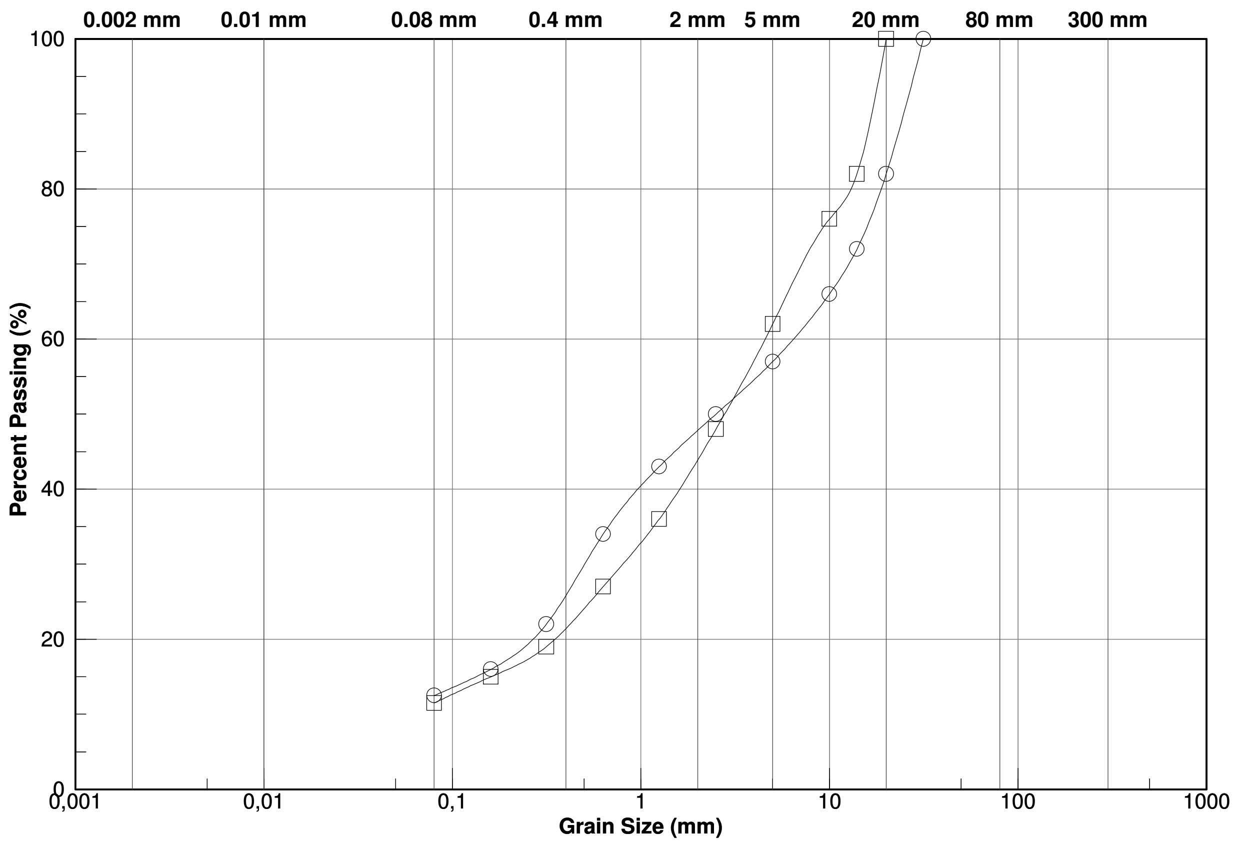
LVM

GRAIN-SIZE ANALYSIS

Project: **Rehabilitation of the Pedestrian Bridge Leamy Creek** Figure n° : **2**

Location: **Route Verte 1, Abutment Northeast, Gatineau, Quebec** File n° : **B-0001957-1**



CLAY	SILT	SAND			GRAVEL		COBBLE	BOULDER
		FINE	MEDIUM	COARSE	FINE	COARSE		

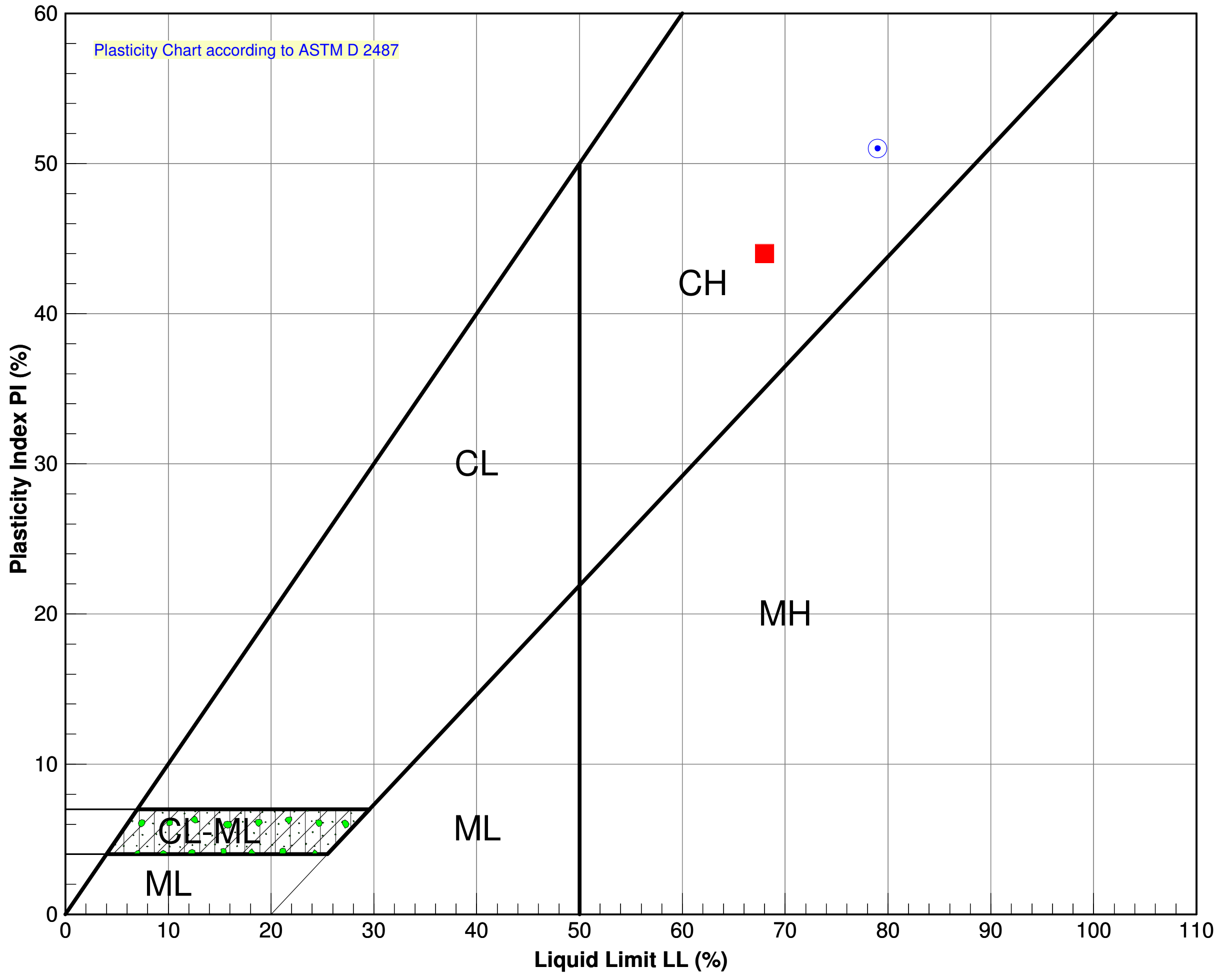
Col. symboles	Borehole n°	Sample n°	Depth (m)	Description	USCS class. (ASTM D-2487)
○	BH-01-12	SS-2	0.76 - 1.37	Sand and gravel with some silt	SM
□	BH-03-12	SS-1	0.08 - 0.61	Sand and gravel with some silt	SP-SM

EQ-09-Ge-68A R.1 04.03.2009

PLASTICITY CHART

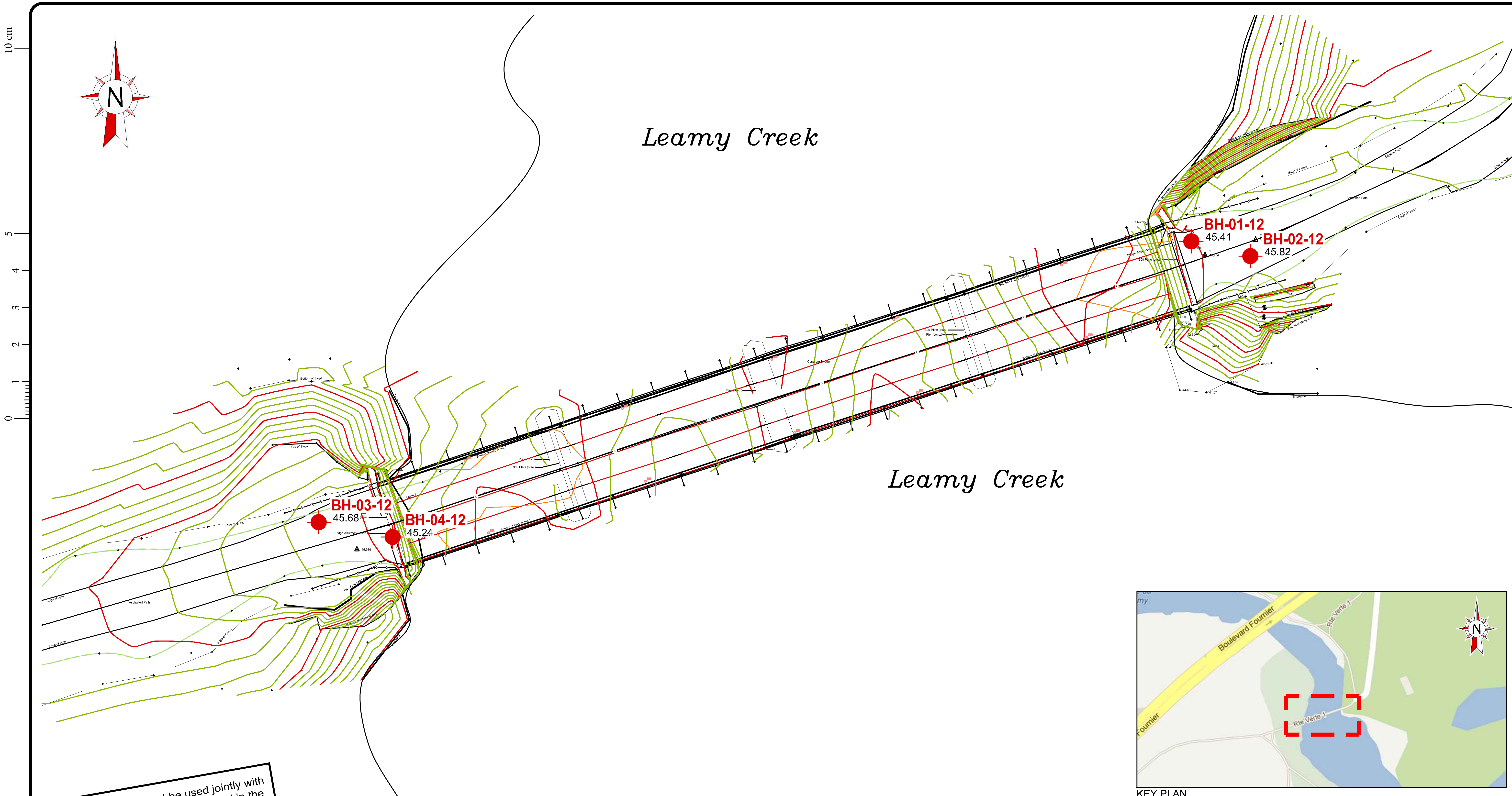
Project : **Rehabilitation of the Pedestrian Bridge Leamy Creek** Figure n° : **3**

Location: **Route Verte 1, Abutment Southeast, Gatineau, Quebec** File n° : **B-0001957-1**



Symbol	Borehole n°	Sample n°	Depth (m)	W	L_L	P_L	P_I	L_L	USCS Class.
■	BH-03-12	SS-13	9.14 - 9.75	69.0	68.0	24.0	44	1.0	CH
⊙	BH-03-12	SS-7	4.57 - 5.18	54.0	79.0	28.0	51	0.5	CH

Appendix 4 Plan of Boreholes Locations



KEY PLAN

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

LEGEND :

BH-NN-YY
00.00 BOREHOLE-NUMBER-YEAR
ELEVATION (m)

BOREHOLES COORDINATES

BOREHOLE	NORTH (y)	EAST (x)	ELEVATION (m)
BH-01-12	5034743	366812	45,41
BH-02-12	5034742	366816	45,82
BH-03-12	5034724	366753	45,68
BH-04-12	5034723	366758	45,24

Project

National Capital Commission
Rehabilitation of the Pedestrian Bridge
Leamy Creek

Title

Borehole Location Plan

LVM

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Prepared S. Séguin
Drawn R. Frenette
Checked T. Lampron

Discipline Geotechnical
Scale 1:250
Date 2013-12-16

Project Manager
Y. Coulibaly
Extract from: Rev.:

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